CHAPTER 5

ENGINEERING CALCULATIONS

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (con't.)

LIST OF ILLUSTRATIONS (con't.)

ENGINEERING CALCULATIONS

5-1 INTRODUCTION

This section provides guidelines for performing various engineering calculations associated with the design of stormwater management facilities such as extended-detention and retention basins and multi-stage outlet structures. The prerequisite information for using these calculations is the determination of the hydrologic characteristics of the contributing watershed in the form of the peak discharge (in *cfs*), or a runoff hydrograph, depending on the hydrologic and hydraulic routing methods used. (Refer to **Section 4-4** in **Chapter 4** for hydrologic methods.)

5-2 GENERAL INFORMATION: DETENTION, EXTENDED-DETENTION AND RETENTION BASIN DESIGN CALCULATIONS

Based on Virginia's Stormwater Management Regulations, a stormwater management basin may be designed to *control water quantity* (for flood control and channel erosion control) and to *enhance* (or treat) *water quality*. The type of basin selected (extended-detention, retention, infiltration, etc.) and the relationship between its design components *(design inflow, storage volume* and *outflow)* will dictate the size of the basin and serve as the basis for its hydraulic design. Some design component parameters such as *design storm return frequency, allowable discharge rates*, etc., may be specified by the local regulatory authority, based upon the specific needs of certain watersheds or stream channels within that locality. Occasionally, as in stream channel erosion control, it may be up to the engineer to document and analyze the specific needs of the downstream channel and establish the design parameters.

The ***design inflow*** is either the *peak flow* or the *runoff hydrograph* from the developed watershed. This *inflow* becomes the input data for the basin sizing calculations, often called ***routings***. Various routing methods are available. Note that the format of the hydrologic input data will usually be dictated by whatever routing method is chosen. (The methods discussed in this handbook require the use of a *peak discharge* or an actual *runoff hydrograph*.) Generally, larger and more complex projects will require a detailed analysis, which includes a runoff hydrograph. Preliminary studies and small projects may be designed using simpler, shortcut techniques that only require a peak discharge. For all projects, the designer must document the hydrologic conditions to support the inflow portion of the hydraulic relationship.

Achieving adequate ***storage volume*** within a basin can usually be accomplished by manipulation of the site grades and strategic placement of the permanent features such as buildings and parking lots. Sometimes, the location of a stormwater facility will be dictated by the site topography and available outfall location. (Refer to **Chapter 3** for basin planning considerations and design criteria.) Storage volume calculations will be discussed in detail later in this chapter.

5-3 ALLOWABLE RELEASE RATES

The allowable release rates for a stormwater facility are dependent on the proposed function(s) of that facility, such as ***flood control***, ***channel erosion control***, or ***water quality enhancement***. For example, a basin used for *water quality enhancement* is designed to detain the ***water quality volume*** and slowly release it over a specified amount of time. This water quality volume is the ***first flush*** of runoff, which is considered to contain the largest concentration of pollutants (Schueler 1987). (Refer to **Section 5-6** for water quality volume calculations.) In contrast, a basin used for *flood* or *channel erosion control* is designed to detain and release runoff from a given storm event at a *predetermined maximum release rate*. This *release rate* may vary from one watershed to another based on predeveloped conditions.

Localities, through stormwater management and erosion control ordinances, have traditionally set the allowable release rates for given frequency storm events to equal the watershed's pre-developed rates. This technique has become a convenient and consistent mechanism to establish the design parameters for a stormwater management facility, particularly as it relates to flood control or stream channel erosion control.

Chapter 4 discusses the impact of development on the hydrologic cycle and the difficulty in re-establishing the pre-developed runoff characteristics. Although it is popular to set a stormwater basin's allowable release rate to the watershed's pre-developed rate, this technique rarely duplicates existing conditions, particularly as it relates to storm frequencies and duration.

In Virginia, the allowable release rate for controlling stream channel erosion or flooding may be established by ordinance using the state's minimum criteria, or by analyzing specific downstream topographic, geographic or geologic conditions to select alternate criteria. **The engineer should be aware of what the local requirements are <u>before</u> designing**.

The design examples and calculations in this handbook use the state minimum requirements for illustrative purposes. **Example 1,** which considers a single homogeneous watershed, is summarized here to show the allowable release rates calculated for the basin. These release rates, as required by the state stormwater regulations, are the pre-developed runoff rates for the 2- and 10-year design storms. **Table 5-1** provides a summary of the hydrologic analysis for **Example 1**. (The complete solution to **Example 1** is provided in **Chapter 6**.)

TABLE 5 - 1
Hydrologic Summary*, *Example 1, SCS Methods

TR-55 GRAPHICAL PEAK DISCHARGE					
CONDITION	**DA**	**RCN**	t_c	Q_2	Q_{10}
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.5 *cfs**	26.8 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	29.9 *cfs*	70.6 *cfs*
TR-20 COMPUTER RUN					
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.0 *cfs**	25.5 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	25.9 *cfs*	61.1 *cfs*

Allowable release rate

5-4 STORAGE VOLUME REQUIREMENT ESTIMATES

Stormwater management facilities are designed using a trial and error process. The designer does many iterative routings to select a minimum facility size with the proper outlet controls. Each iterative routing requires that the facility size (*stage-storage relationship*) and the outlet configuration (*stage-discharge relationship*) be evaluated for performance against the watershed requirements. A graphical evaluation of the ***inflow hydrograph*** versus an approximation of the ***outflow rating curve*** provides the designer with an estimate of the required ***storage volume***. Starting with this **assumed** required volume, the number of iterations is reduced.

The ***graphical hydrograph analysis*** requires that the evaluation of the watershed's hydrology produce a runoff hydrograph for the appropriate design storms. The state stormwater management regulations allow the use of SCS methods or the modified rational method (critical storm duration approach) for analysis. Many techniques are available to generate the resulting runoff hydrographs based on these methods. It is the designer's responsibility to be familiar with the limitations and assumptions of the methods as they apply to generating hydrographs (refer to **Chapter 4, Hydrologic Methods**).

Graphical procedures can be time consuming, especially when dealing with multiple storms, and are therefore not practical when designing a detention facility for a small site development. Shortcut procedures have been developed to allow the engineer to approximate the storage volume requirements. Such methods include **TR-55: Storage Volume for Detention Basins**, **Section 5-4.2**, and **Critical Storm Duration-Modified Rational Method-Direct Solution**, **Section 5-4.4**, which can be used as planning tools. Final design should be refined using a more accurate hydrograph routing procedure. Sometimes, these shortcut methods may be used for final design, but they must be used with caution since they only approximate the required storage volume (refer to the assumptions and limitations for each method).

It should be noted that the TR-55 tabular hydrograph method does not produce a full hydrograph. The tabular method generates only the portion of the hydrograph that contains the peak discharge and some of the time steps just before and just after the peak. The missing values must be extrapolated, thus potentially reducing the accuracy of the hydrograph analysis. It is recommended that if SCS methods are to be used, a full hydrograph be generated using one of the available computer programs. The accuracy of the analysis can only be as accurate as the hydrograph used.

5-4.1 Graphical Hydrograph Analysis - SCS Methods

The following procedure represents a graphical hydrograph analysis that results in the approximation of the required storage volume for a proposed stormwater management basin. **Example 1** is presented here to illustrate this technique. See **Table 5-1** for a summary of the hydrology. The TR-20 computer-generated inflow hydrograph is used for this example. The allowable discharge from the proposed basin has been established by ordinance (based on pre-developed watershed discharge).

Information Needed:

The pre- and post-developed hydrology, which includes the pre-developed peak rate of runoff ***(allowable release rate***) and the post-developed runoff hydrograph (***inflow hydrograph***) is required for hydrograph analysis (see **Table 5-1**).

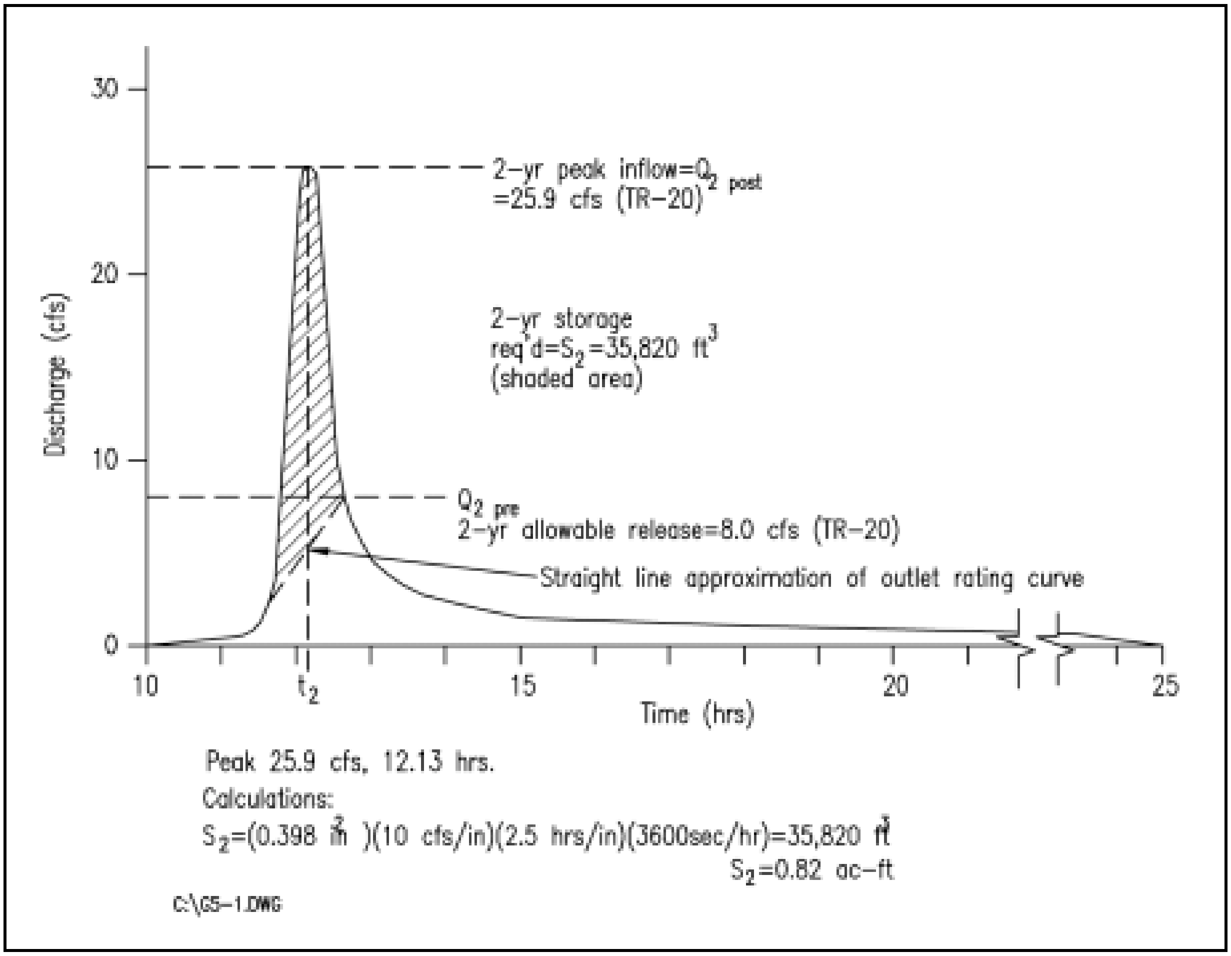
Procedure

(Refer to **Figure 5-1** for the 2-year developed inflow hydrograph and **Figure 5-2** for the 10-year developed inflow hydrograph):

1. Commencing with the plot of the 2-year developed inflow hydrograph (Discharge vs. Time), the 2-year allowable release rate, $Q_2 = 8\ cfs$, is plotted as a horizontal line starting at time $t = 0$ and continuing to the point where it intersects the falling limb of the hydrograph.

2. A diagonal line is then drawn from the beginning of the inflow hydrograph to the intersection point described above. This line represents the ***hypothetical rating curve*** of the control structure and approximates the rising limb of the outflow hydrograph for the 2-year storm.

FIGURE 5 - 1
SCS Runoff Hydrograph, Example 1, 2-Year Post-Developed



3. The ***storage volume*** is then approximated by calculating the area under the inflow hydrograph, less the area under the rising limb of the outflow hydrograph. This is shown as the shaded area in **Figure 5-1**. The storage volume required for the 2-year storm, S_2, can be approximated by measuring the shaded area with a planimeter.

The vertical scale of a hydrograph is in cubic feet per second (*cfs*) and the horizontal scale is in hours (hrs). Therefore, the area, as measured in square inches (*in^2*), is multiplied by scale conversion factors of *cfs* per inch, hours per inch, and 3600 seconds per hour, to yield an area in cubic feet (*ft^3*). The conversion is as follows:

$$
\begin{aligned}
S_2 &= (0.398\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\
&= 35{,}820\ ft^3 \\
&= 0.82\ ac.ft.
\end{aligned}
$$

4. On a plot of the 10-year inflow hydrograph, the 10-year allowable release rate, Q_{10} , is plotted as a horizontal line extending from time zero to the point where it intersects the falling limb of the hydrograph.

5. By trial and error, the time t_2 , at which the S_2 volume occurs while maintaining the 2-year release, is determined by planimeter. This is represented by the shaded area to the left of t_2 on **Figure 5-2**. From the intersection point of t_2 and the 2-year allowable release rate, Q_2 , a line is drawn to connect to the intersection point of the 10-year allowable release rate and the falling limb of the hydrograph. This intersection point is t_{10} , and the connecting line is a straight line approximation of the *outlet rating curve*.

6. The area under the inflow hydrograph from time t_2 to time t_{10} , less the area under the rising limb of the hypothetical rating curve, represents the additional volume (shaded area to the right of t_2 on **Figure 5-2**) needed to meet the 10-year storm storage requirements.

7. The total storage volume, S_{10} , required, can be computed by adding this additional storage volume to S_2 . This is represented by the total shaded area under the hydrograph.

$$\begin{aligned} S_{10} &= (0.89\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\ &= 80{,}100\ ft^3 \\ &= 1.84\ ac.ft. \end{aligned}$$

These steps may be repeated if storage of the 100-year storm, or any other design frequency storm, is required by ordinance or downstream conditions.

In summary, the total volume of storage required is the area under the runoff hydrograph curve and above the basin outflow curve. It should be noted that the outflow rating curve is approximated as a straight line. The actual shape of the outflow rating curve will depend on the type of outlet device used. **Figure 5-3** shows the typical shapes of outlet rating curves for orifice and weir outlet structures. The straight line approximation is reasonable for an orifice outlet structure. However, this approximation will likely **underestimate** the storage volume required when a weir outlet structure is used. Depending on the complexity of the design and the need for an exact engineered solution, the use of a more rigorous sizing technique, such as a storage indication routing, may be necessary.

FIGURE 5 - 2
SCS Runoff Hydrograph, Example 1, 10-Year Post-Developed

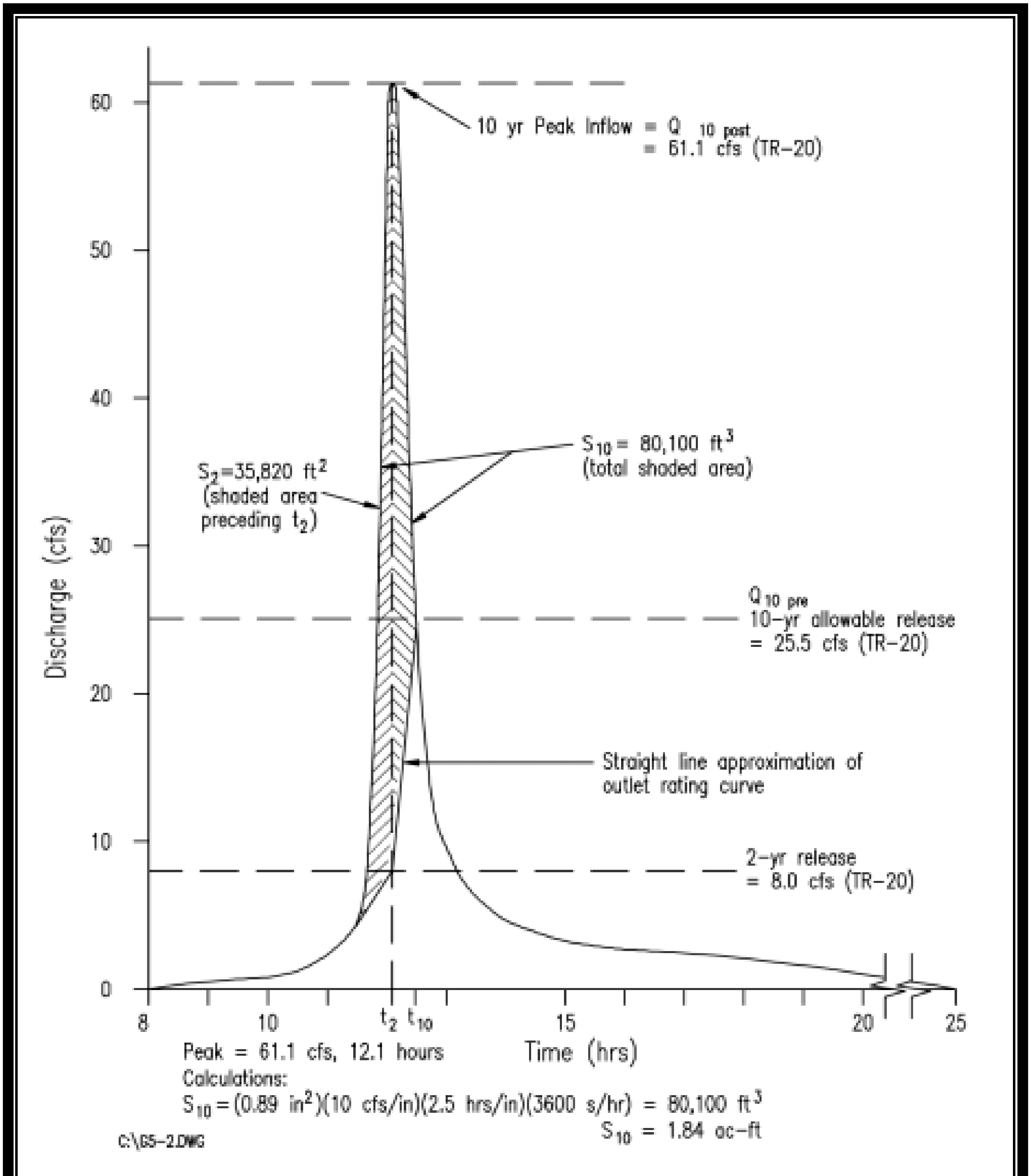
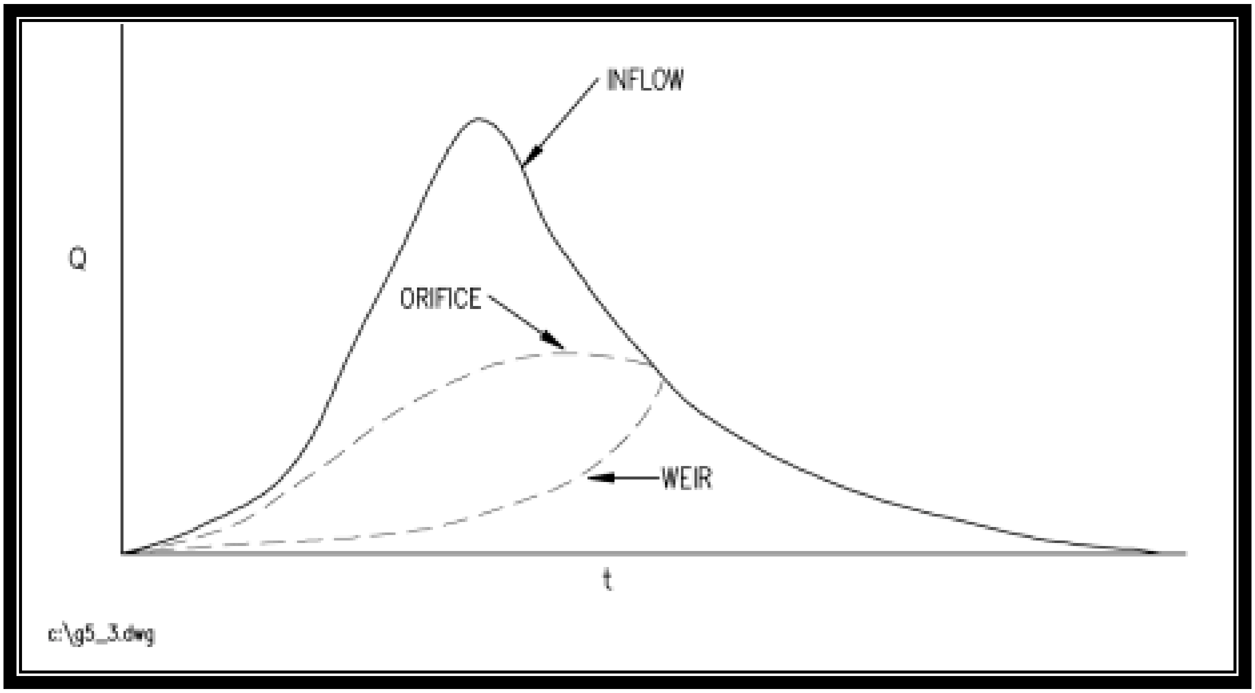


FIGURE 5 - 3

Typical Outlet Rating Curves for Orifice and Weir Outlet Devices

5-4.2 TR-55: Storage Volume for Detention Basins (Shortcut Method)



The TR-55 Storage Volume for Detention Basins, or TR-55 shortcut procedure, provides similar results to the graphical analysis described in **Section 5-4.1**. This method is based on average storage and routing effects for many structures. TR-55 can be used for single-stage or multi-stage outflow devices. The only constraints are that 1) *each stage requires a design storm and a computation of the storage required for it*, and 2) *the discharge of the upper stage(s) includes the discharge of the lower stage(s)*. Refer to TR-55 for more detailed discussions and limitations.

Information Needed:

To calculate the required storage volume using TR-55, the pre- and post-developed hydrology per SCS methods is needed (refer to **Chapter 4**). This includes the watershed's *pre-developed peak rate of discharge*, or *allowable release rate*, Q_o , the watershed's *post-developed peak rate of discharge*, or *inflow*, Q_i , for the appropriate design storms, and the watershed's *post-developed runoff*, *Q*, in inches. (Note that this method does **not** require a hydrograph.)

Once the above parameters are known, the TR-55 Manual can be used to approximate the storage volume required for each design storm. The following procedure summarizes the TR-55 shortcut method using the 25-acre watershed presented in **Example 1**.

Procedure:

1. Determine the peak developed inflow, Q_i, and the allowable release rate, Q_o, from the hydrology for the appropriate design storm. The 2-year storm flow rates from Example 1 (TR-55 Graphical peak discharge) are used here:

$$Q_{o_2} = 8.5\ cfs\ ; \quad Q_{i_2} = 29.9\ cfs$$

Using the ratio of the allowable release rate, Q_o , to the peak developed inflow, Q_i , or Q_o/Q_i , for the appropriate design storm, use **Figure 5-4** (or Figure 6-1 in TR-55) to obtain the ratio of storage volume, V_s, to runoff volume, V_r , or V_s/V_r .

From **Example 1**:

$$Q_{o_2}/Q_{i_2} = 8.5 / 29.9 = 0.28$$

From **Figure 5-4** or TR-55 Figure 6.1:

V_{s_2} / V_{r_2};39

2. Determine the runoff volume, V_r , in *ac.ft.*, from the TR-55 worksheets for the appropriate design storm.

$$V_r = Q\ A_m\ 53.33$$

where:

Q	=	*runoff, in inches, from TR-55 Worksheet 2*
A_m	=	*drainage area, in square miles*
53.33	=	*conversion factor to acre-feet*

From **Example 1**:

$$Q_2 = 1.30\ in.$$
$$A_m = 25\ ac. / 640\ ac./mi^2 = 0\ .039\ mi^2$$

$$V_{r_2} = 1.30(.039)\ 53.33$$
$$= 2.70\ ac.ft.$$

3. Multiply the V_s/V_r ratio from Step 1 by the runoff volume, V_r, from Step 2, to determine the volume of storage required, V_s, in acre-feet.

$$\left(\frac{V_s}{V_r}\right)V_r \qquad V_s$$

From **Example 1**:

$$(.39)(2.70 \text{ ac.ft.}) = 1.05 \text{ ac.ft.}$$

4. Repeat these steps for each additional design storm as required to determine the approximate storage requirements. The 10-year storm storage requirements from **Example 1** are presented here:

   a. $Q_o = 26.8$ *cfs*
      $Q_i = 70.6$ *cfs*

      $Q_o/Q_i = 26.8/70.6 = 0.38$; From **Figure 5-4** or TR-55 Figure 6-1: $V_s/V_r = .33$

   b. $V_r = QA_m\, 53.33 = 2.85$ *in.*$(.039$ *sq.mi.*$)(53.33) = 5.93$ *ac.ft.*

   c. $V_s = (V_s/V_r)V_r = (.33)\ 5.93$ *ac.ft.* $= 1.96$ *ac.ft.*

This volume represents the total storage required for the 2-year storm and the 10-year storm.

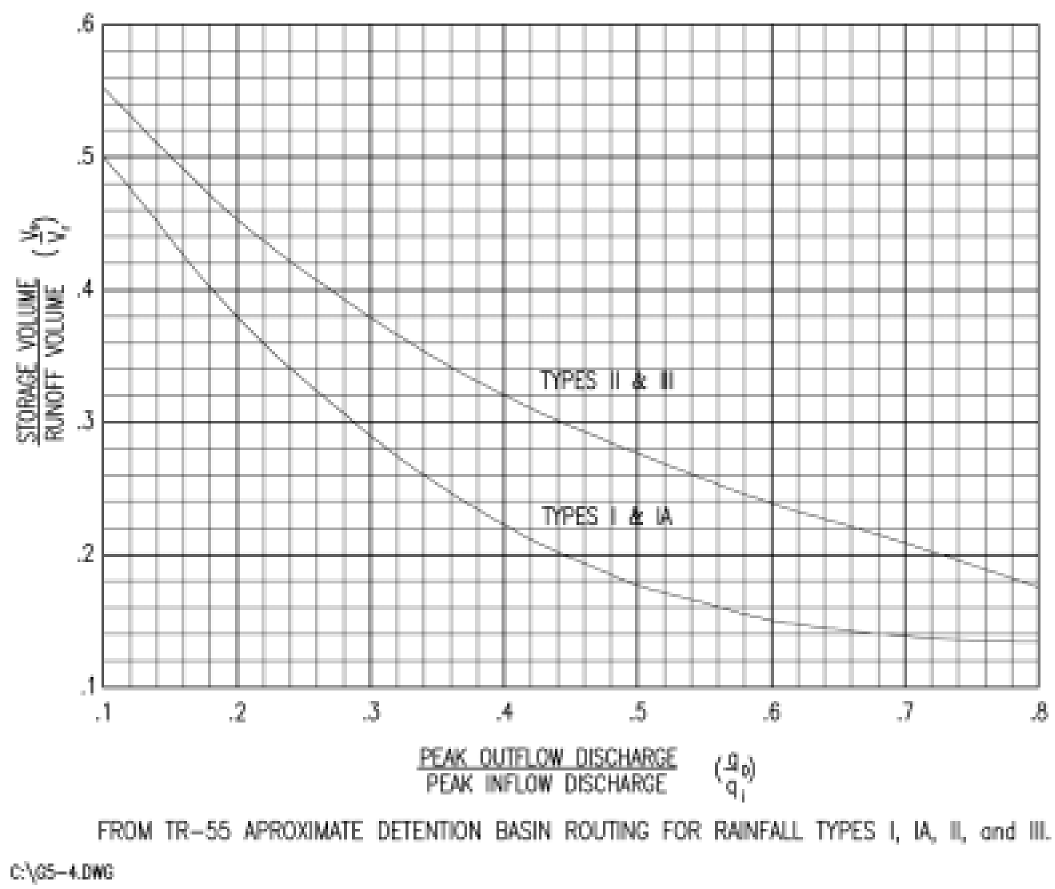
5. NOTE: The volume from **#4** above may need to be increased if additional storage is required for water quality purposes or channel erosion control. Refer to **Section 5-6** or **Section 5-10**, respectively.

The design procedure presented above should be used with TR-55 Worksheet 6a, as shown in **Example 1** of **Chapter 6**. The worksheet includes an area to plot the *stage-storage curve*, from which actual elevations corresponding to the required storage volumes can be derived. **Table 5-2** provides a summary of the required storage volumes using the graphical SCS hydrograph analysis and the TR-55 shortcut method.

TABLE 5 - 2
Storage Volume Requirements, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	0.82 *ac.ft.*	1.84 *ac.ft.*
TR-55 Shortcut Method	1.05 *ac.ft.*	1.96 *ac.ft.*

FIGURE 5 - 4
Approximate Detention Basin Routing for Rainfall Types I, IA, II and III



Source: SCS TR-55 Urban Hydrology for Small Watersheds: Figure 6-1

5-4.3 Graphical Hydrograph Analysis, Modified Rational Method - Critical Storm Duration

The Modified Rational Method uses the ***critical storm duration*** to calculate the ***maximum storage volume*** for a detention facility. This *critical storm duration* is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. In contrast, the Rational Method produces a triangular runoff hydrograph that gives the peak inflow at time = t_c and falls to zero flow at time = $2.5t_c$. In theory, this hydrograph represents a storm whose duration equals the time of concentration, t_c, resulting in the greatest peak discharge for the given return frequency storm. The volume of runoff, however, is of greater consequence in sizing a detention facility. A storm whose duration is longer than the t_c may not produce as large a peak rate of runoff, but it may generate a greater **volume** of runoff. By using the Modified Rational Method, the designer can evaluate several different storm durations to verify which one requires the greatest volume of storage with respect to the allowable release rate. It is this *maximum storage volume* that the basin must be designed to detain.

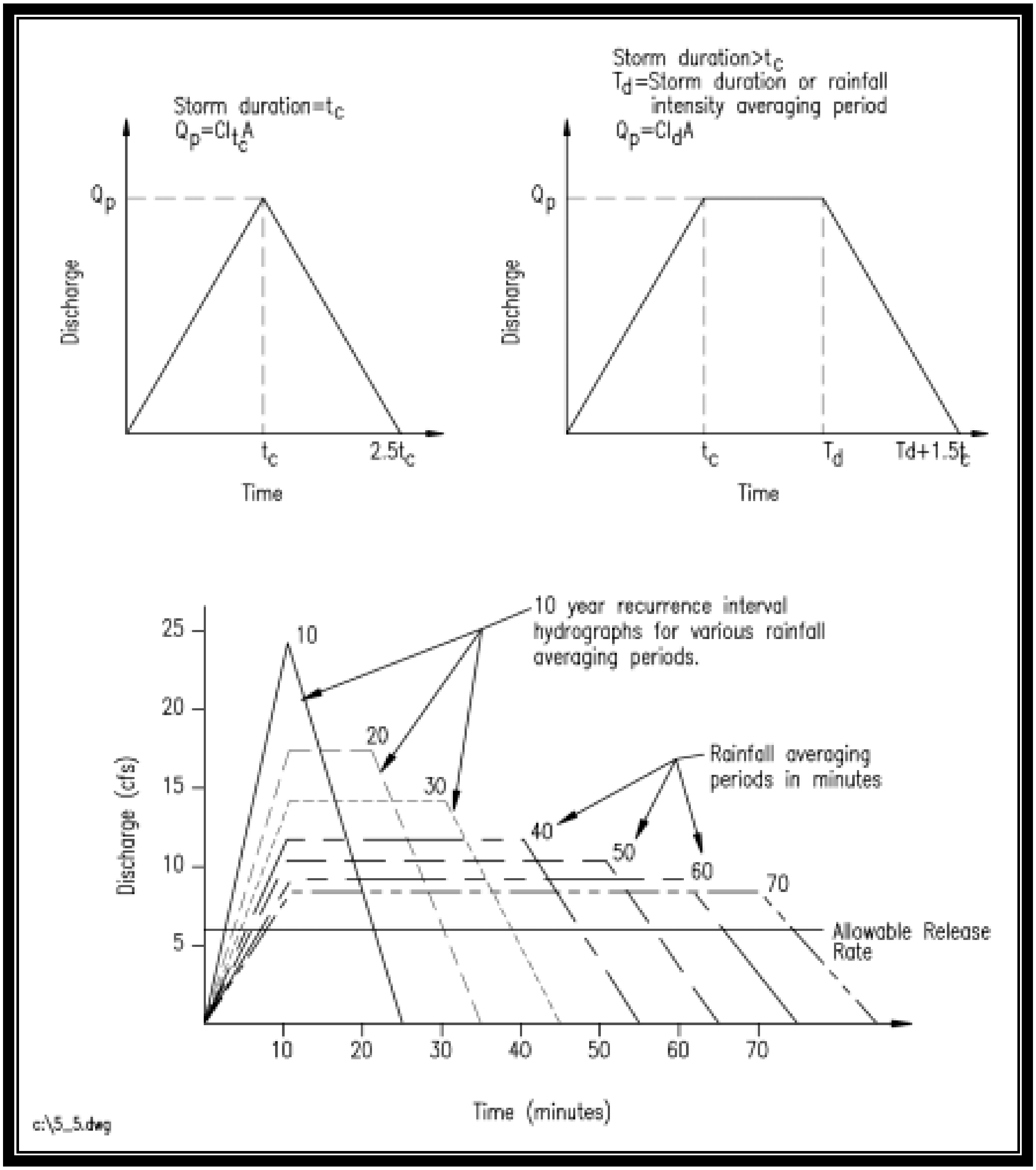
The first step in determining the critical storm duration is to use the post-developed time of concentration, t_c, to generate a post-developed runoff hydrograph. Rainfall intensity averaging periods, T_d, representing time periods incrementally longer than the t_c, are then used to generate a "family" of runoff hydrographs for the same drainage area. These hydrographs will be trapezoidal with the peak discharges, Q_i, based upon the intensity, I, of the averaging period, T_d. **Figure 5-5** shows the construction of a typical triangular and trapezoidal hydrograph using the modified rational method, and a family of trapezoidal hydrographs representing storms of different durations.

Note that the duration of the receding limb of the trapezoidal hydrograph, in **Figure 5-5**, is set to equal 1.5 times the time of concentration, t_c. Also, the total hydrograph duration is $2.5t_c$ versus $2t_c$ as discussed in **Chapter 4**. This longer duration is considered more representative of actual storm and runoff dynamics. It is also more analogous to the SCS unit hydrograph where the receding limb extends longer than the rising limb.

The Modified Rational Method assumes that the rainfall intensity averaging period is equal to the actual storm duration. This means that the rainfall and runoff that occur before and after the rainfall averaging period are not accounted for. **Therefore, the Modified Rational Method may underestimate the required storage volume for any given storm event**.

The rainfall intensity averaging periods are chosen arbitrarily. However, the designer should select periods for which the corresponding intensity-duration-frequency (I-D-F) curves are available, i.e., 10 *min.*, 20 *min.*, 30 *min.*, etc. The shortest period selected should be the time of concentration, t_c. A straight line starting at $Q = 0$ and $t = 0$ and intercepting the inflow hydrograph on the receding limb at the allowable release rate, Q_o, represents the outflow rating curve. The time averaging period hydrograph that represents the greatest storage volume required is the one with the largest area between the inflow hydrograph and outflow rating curve. This determination is made by a graphical analysis of the hydrographs.

FIGURE 5 - 5
Modified Rational Method Hydrographs



The following procedure represents a graphical analysis very similar to the one described in **Section 5-4.1**. **Example 1** from **Chapter 6** will be used again. Note that **the rational and modified rational methods should normally be used in homogeneous drainage areas of less than 20 acres, with a t_c of less than 20 minutes**. Although the watershed in **Example 1** has a drainage area of 25 acres and a t_c of greater than 20 minutes, it will be used here for illustrative purposes. Note that the pre- and post-developed peak discharges are much greater than those calculated using the SCS method applied to the same watershed. This difference may be the result of the large acreage and t_c values.

A summary of the hydrology is found in **Table 5-3**. Note that the t_c calculations were performed using the more rigorous SCS TR-55 method.

TABLE 5 - 3
Hydrologic Summary, Example 1, Rational Method

Rational Method					
CONDITION	**D.A.**	***C***	***T_c***	***Q_2***	***Q_{10}***
Pre-developed	*25 ac.*	*.38*	*.87 hr* *52 min.*	*17 cfs*	*24 cfs*
Post-developed	*25 ac.*	*.59*	*.35 hr.* *21 min.*	*49 cfs*	*65 cfs*

Information Needed:

The Modified Rational Method-Critical Storm Duration Approach is very similar to SCS methods since it requires pre- and post-developed hydrology in the form of a pre-developed peak rate of runoff (***allowable release rate)*** and a post-developed runoff hydrograph (***inflow hydrograph***), as developed using the Rational Method.

Procedure: (Refer to **Figures 5-6** and **5-7**.)

1. Plot the 2-year developed condition inflow hydrograph (triangular) based on the developed condition, t_c.

2. Plot a family of hydrographs, with the time averaging period, T_d, of each hydrograph increasing incrementally from 21 minutes (developed condition t_c) to 60 minutes, as shown in **Figure 5-6**. Note that the first hydrograph is a Type 1 Modified Rational Method triangular hydrograph, as shown in **Figure 4-7** in **Chapter 4,** where the storm duration, *d*, or T_d, is equal

to the time of concentration, t_c . The remaining hydrographs are trapezoidal, or Type 2 hydrographs. The peak discharge for each hydrograph is calculated using the rational equation, $Q = CIA$, where the intensity, I, from the I-D-F curve is determined using the rainfall intensity averaging period as the storm duration.

3. Superimpose the outflow rating curve on each inflow hydrograph. The area between the two curves then represents the storage volume required, as shown in **Figure 5-6**. Similar cautions, as described in the SCS Methods, **Section 5-4.1**, regarding the straight line approximation of the outlet discharge curve apply here as well. The actual shape of the outflow curve depends on the type of outlet device.

4. Compute and tabulate the required storage volume for each of the selected rainfall durations or time averaging periods, T_d, using the procedures described in **Section 5-4.1**.

   The storm duration that requires the maximum storage is the *critical storm* and is used for the sizing of the basin. (A storm duration equal to the t_c produces the largest storage volume required for the 2-year storm presented here.)

5. Repeat Steps 1 through 4 above for the analysis of the 10-year storage requirements. (**Figure 5-7 represents this procedure repeated for the 10-year design storm**.)

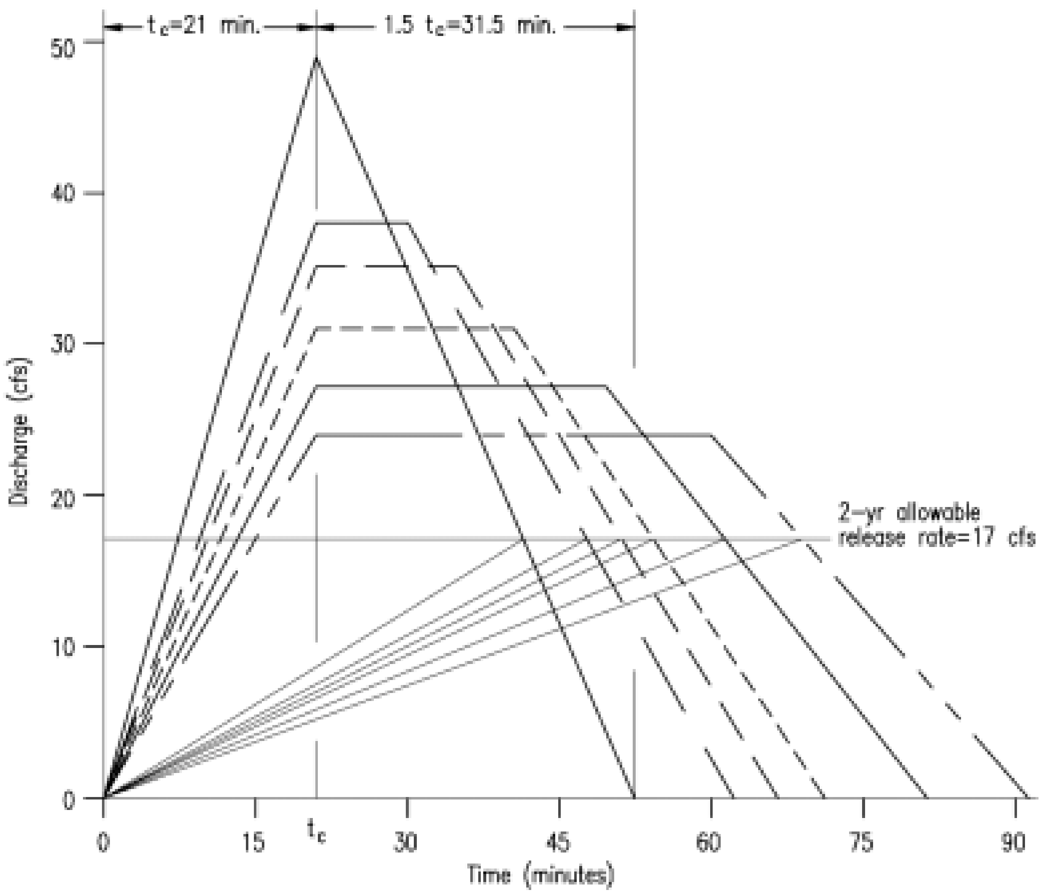
Conveyance systems should still be designed using the Rational Method, as opposed to the Modified Rational Method, to ensure their design for the peak rate of runoff.

TABLE 5 - 4

Storage Volume Requirements - Example 1

Method	2-*yr.* Storage Required	10-*yr.* Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration	*1.16 ac.ft.* T_d *= 21 min.*	*1.56 ac.ft.* T_d *= 40 min.*

FIGURE 5 - 6
Modified Rational Method Runoff Hydrograph, Example 1, 2-Year Post-Developed Condition



Conversion – in^2 to ft^3 = 10 cfs/in x 0.25 hrs/in x 3600 sec/hr=9000 ft^3/in^2

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in^2)	Storage Volume (ft^3)
21 min	3.3	49	5.62	50,580*
30 min	2.6	38	5.51	49,590
35 min	2.4	35	5.53	49,770
40 min	2.1	31	5.29	47,610
50 min	1.8	27	5.25	47,250
60 min	1.6	24	5.24	47,160

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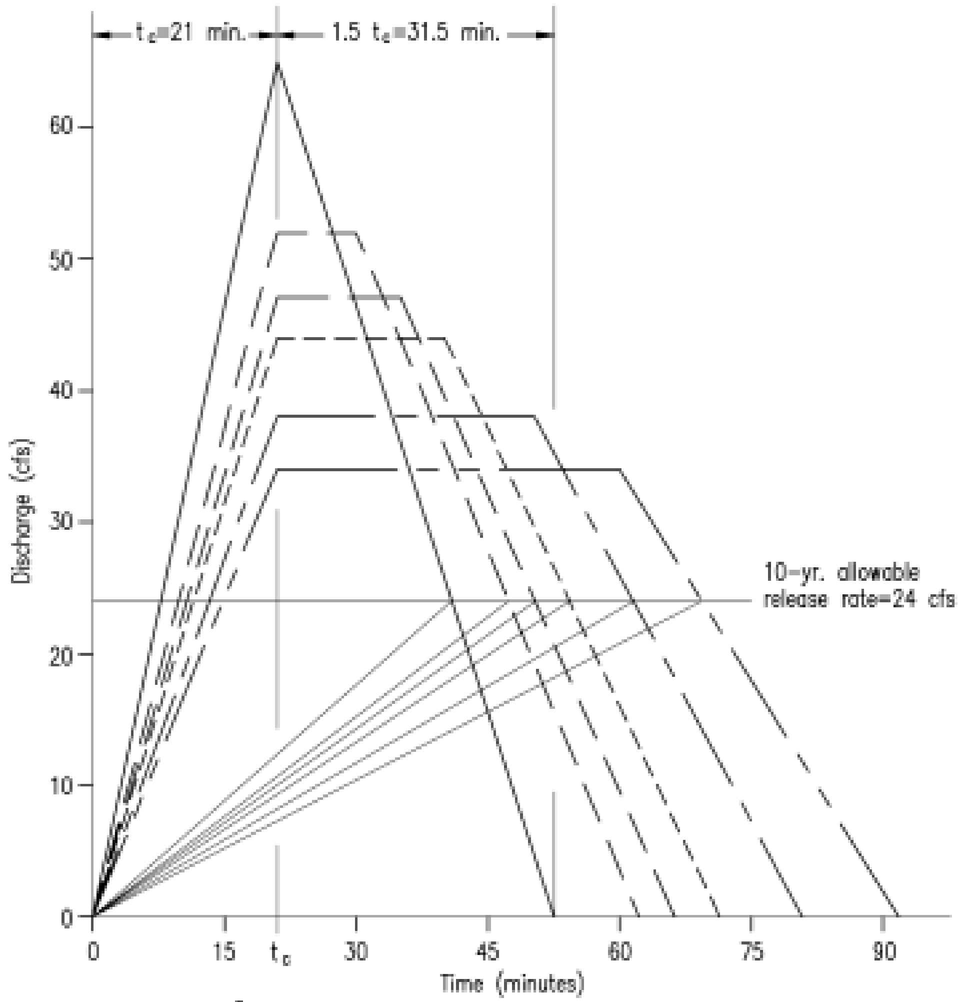
FIGURE 5 - 7
Modified Rational Method Runoff Hydrograph, Example 1,
10-Year Post-Developed Condition



Conversion = in² to ft³= 9000 ft³/in²

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in²)	Storage Volume (ft³)
21 min	4.4	65	7.07	63,630
30 min	3.5	52	7.33	65,970
35 min	3.2	47	7.31	65,790
40 min	3.0	44	7.53	67,770*
50 min	2.6	38	7.45	67,050
60 min	2.3	34	7.48	67,320

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5-4.4 Modified Rational Method, Critical Storm Duration - Direct Solution

A direct solution to the Modified Rational Method, Critical Storm Duration has been developed to eliminate the time intensive, iterative process of generating multiple hydrographs. This direct solution takes into account the storm duration and allows the designer to solve for the time at which the storage volume curve has a slope equal to zero, which corresponds to maximum storage. The basic derivation of this method is provided below, followed by the procedure as applied to **Example 1**.

Storage Volume

The runoff hydrograph developed with the Modified Rational Method, Critical Storm Duration will be either triangular or trapezoidal in shape. The outflow hydrograph of the basin is approximated by a straight line starting at *0 cfs* at the time *t=0* and intercepting the receding leg of the runoff hydrograph at the allowable discharge, q_o.

> ***The straight line representation of the outflow hydrograph is a conservative approximation of the shape of the outflow hydrograph for an orifice control release structure. This method should be used with caution when designing a weir control release structure.***

The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph in **Figure 5-8**. This area can be approximated using the following equation:

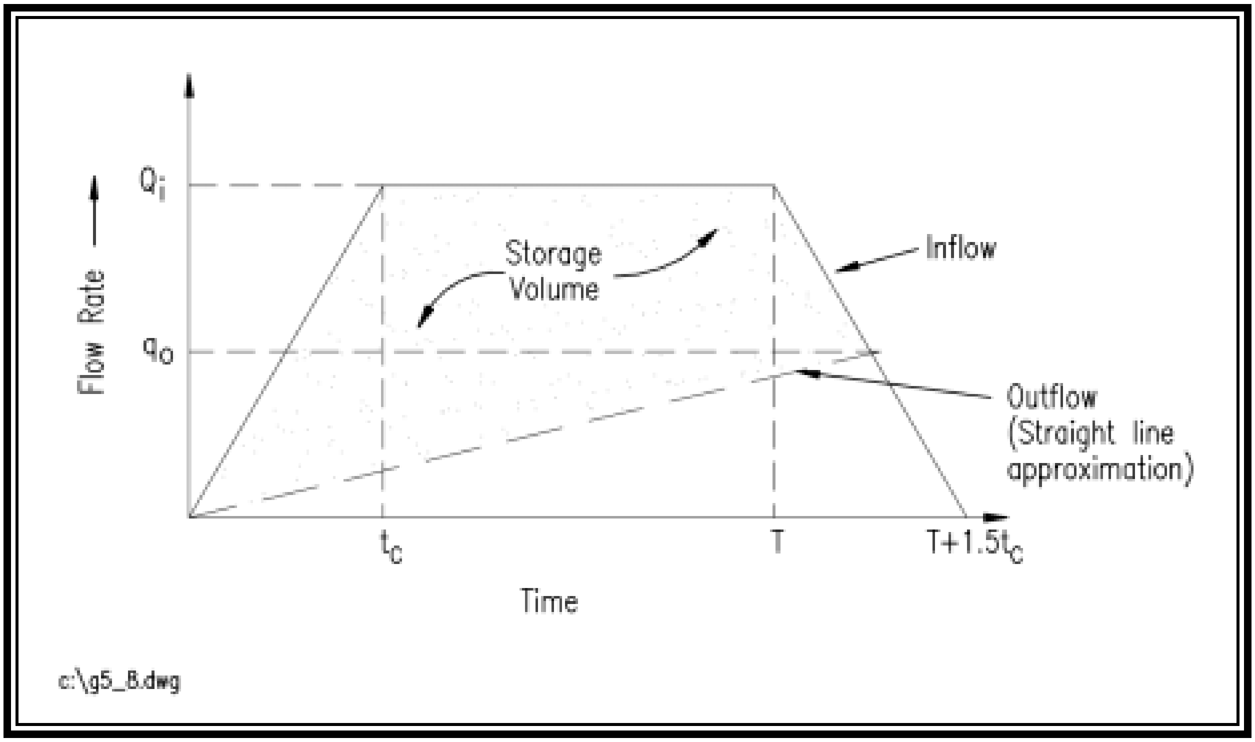
$$V \qquad \left[Q_i T_d \qquad \frac{Q_i t_c}{4} \qquad \frac{q_o T_d}{2} \qquad \frac{3q_o t_c}{4}\right] 60$$

Equation 5-1
Trapezoidal Hydrograph Storage Volume Equation

Where:
- V = *required storage volume, ft*^{3}
- Q_i = *inflow peak discharge, cfs, for the critical storm duration,* T_d
- t_c = *post-developed time of concentration, min.*
- q_o = *allowable peak outflow, cfs*
- T_d = *critical storm duration, min.*

The allowable peak outflow is established by ordinance or downstream conditions. The ***critical storm duration***, T_d, is an unknown and must be determined to solve for the intensity, *I*, and to ultimately calculate the peak inflow, Q_i. Therefore, a relationship between rainfall intensity, *I*, and the critical storm duration, T_d, must be established.

FIGURE 5 - 8
Trapezoidal Hydrograph Storage Volume Estimate



Rainfall Intensity

The rainfall intensity as taken from the I-D-F curves is dependent on the time of concentration, t_c, of a given watershed. Setting the storm duration, T_d, equal to the time of concentration, t_c, will provide the maximum peak discharge. As stated previously, however, it does not necessarily generate the maximum ***volume*** of discharge. Since this maximum volume of runoff is of interest, and the storm duration is unknown, the rainfall intensity, *I*, must be represented as a function of *time*, *frequency*, and *location*. The relationship is expressed as follows:

$$I \qquad \frac{a}{b \;\; T_d}$$

Equation 5-2
Modified Rational Method Intensity, (*I*), Equation

where: I = *rainfall intensity, in./hr.*
T_d = *rainfall duration or rainfall intensity averaging period, min.*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***

TABLE 5 - 5
Rainfall Constants for Virginia*

Duration - 5 minutes to 2 hours			
Station	**Rainfall Frequency**	**Constants**	
Wytheville	2 5 10	117.7 168.6 197.8	19.1 23.8 25.2
Lynchburg	2 5 10	118.8 158.9 189.8	17.2 20.6 22.6
Richmond	2 5 10	130.3 166.9 189.2	18.5 20.9 22.1
Norfolk	2 5 10	126.3 173.8 201.0	17.2 22.7 23.9
Cape Henry	2 5 10	143.2 173.9 203.9	21.0 22.7 24.8

The above constants are based on linear regression analyses of the frequency intensity-duration curves contained in the VDOT Drainage Manual. (Adapted from DCR Course "C" Training Notebook.)

***For a more comprehensive list, see Appendix 5A.**

The rainfall constants, ***a*** and ***b***, were developed from linear regression analyses of the I-D-F curves and can be generated for any area where such curves are available. The limitations associated with the I-D-F curves, such as duration, return frequency, etc., will also limit development of the constants. **Table 5-5** provides rainfall constants for various regions in Virginia. Substituting **Equation 5-2** into the rational equation results in the following:

$$Q \quad C \left(\frac{a}{b \quad T_d} \right) A$$

Equation 5-3
Rearranged Rational Equation

where: Q = *peak rate of discharge, cfs*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*

Substituting this relationship for Q_i , **Equation 5-1** then becomes:

$$V \quad \left[\left[C \left(\frac{a}{b \quad T_d} \right) A \right] T_d \quad \frac{\left[C \left(\frac{a}{b \quad T_d} \right) A \right] t_c}{4} \quad \frac{q_o T_d}{2} \quad \frac{3 q_o t_c}{4} \right] 60$$

Equation 5-4
Substitute *Equation 5-3* into *Equation 5-1*

Maximum Storage Volume

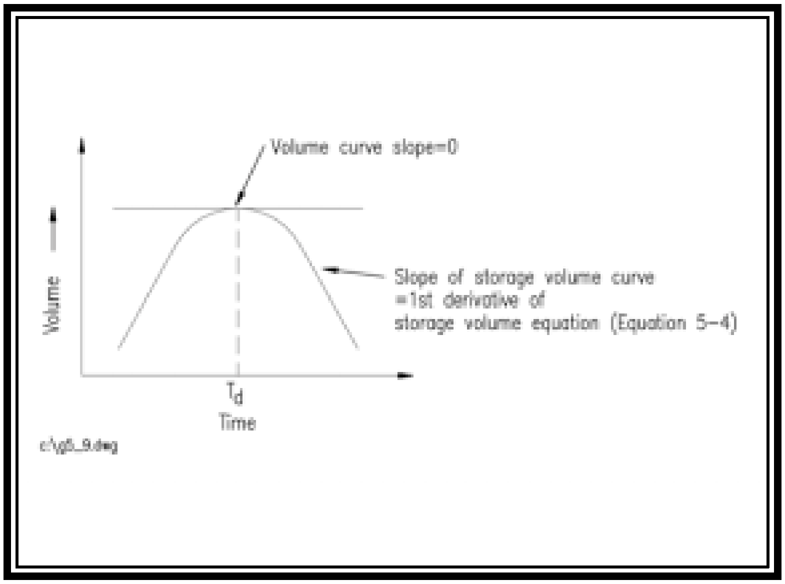
The first derivative of this storage volume equation, **Equation 5-4,** with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When this equation is set to equal zero, and solved for T_d, it represents the time, T_d, at which the slope of the storage volume curve is zero, or at a maximum, as shown in **Figure 5-9**. **Equation 5-5** represents the first derivative of the storage volume equation with respect to time and can be solved for T_d.

$$T_d \quad \sqrt{\frac{2CAa(b \quad t_c/4)}{q_o}} \quad b$$

Equation 5-5
Critical Storm Duration, T_d

where: T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
t_c = *time of concentration, min.*
q_o = *allowable peak outflow, cfs*

FIGURE 5 - 9
Storage Volume vs. Time Curve



Equation 5-5 is solved for T_d. Then, T_d is substituted into **Equation 5-3** to solve for Q_i, and Q_i is substituted into **Equation 5-1** to solve for the required storage volume. Once the storage volume is known, the outlet structure and the stormwater facility can be sized. This method provides a direct solution to the graphical analysis of the family of hydrographs described in **Section 5-4.3** and is quicker to use. The following procedure illustrates this method using **Example 1**:

<u>Information Needed:</u>

The Modified Rational Method-Direct Solution is similar to the previous methods since it requires determination of the pre- and post-developed hydrology, as described in **Section 4-3.1**, resulting in a pre-developed peak rate of runoff (***allowable release rate***) and a post-developed ***runoff hydrograph***. **Table 5-3** provides a summary of the hydrology from **Example 1**. The rainfall constants ***a*** and ***b*** for the watershed are determined from **Table 5-5**.

Procedure:

1. Determine the 2-year critical storm duration by solving **Equation 5-5**:

$$T_{d_2} \quad \sqrt{\frac{2CAa(b \quad t_c/4)}{q_{o_2}}} \quad b$$

Where, from **Example 1**:

T_{d_2} = 2-year critical storm duration, min.
C = developed condition runoff coefficient = .59
A = drainage area = 25.0 acres
t_c = post-developed time of concentration = 21 min.
q_{o_2} = allowable peak outflow = 17 cfs (pre-developed peak rate of discharge)
a_2 = 2-year rainfall constant = 130.3
b_2 = 2-year rainfall constant = 18.5

$$T_{d_2} \quad \sqrt{\frac{2(.59)(25.0)(130.3)(18.5 \quad 21/4)}{17}} \quad 18.5$$

$$\sqrt{2995.9} \quad 18.5$$

$$T_{d_2} \quad 36.2 \ \text{min.}$$

2. Solve for the 2-year critical storm duration intensity, I_2 , using **Equation 5-2** and the 2-year critical storm duration T_{d_2}:

$$I_2 \quad \frac{a}{b \quad T_{d_2}}$$

where: T_{d_2} = *critical storm duration = 36.2 min.*
a = *2-year rainfall constant = 130.3*
b = *2-year rainfall constant = 18.5*

$$I_2 \quad \frac{130.3}{18.5 \quad 36.2} \quad 2.38 \text{ in./hr.}$$

3. Determine the 2-year peak inflow, Q_{i_2}, using the **Rational Equation** and the critical storm duration intensity I_2:

$$Q_{i_2} = CI_2A$$

where: Q_{i_2} = *2-year peak inflow, cfs*
C = *developed condition runoff coefficient = .59*
I_2 = *critical storm intensity = 2.38 in./hr.*
A = *drainage area = 25 acres*

$$Q_{i_2} = (0.59)(2.38)(25)$$

$$Q_{i_2} = 35.1 \text{ cfs}$$

4. Determine the 2-year required storage volume for the 2-year critical storm duration, T_{d_2}, using **Equation 5-1**:

$$V_2 \quad \left[Q_{i_2}T_{d_2} \quad \frac{Q_{i_2}t_c}{4} \quad \frac{q_{o_2}T_{d_2}}{2} \quad \frac{3q_{o_2}t_c}{4} \right] 60$$

where: V_2 = *2-year required storage, ft*^{3}
Q_{i_2} = *2-year peak inflow for critical storm = 35.1 cfs*
C = *developed runoff coefficient = .59*
A = *area = 25.0 acres*
T_{d_2} = *critical storm duration = 36.2 min.*
t_c = *developed condition time of concentration = 21 min.*
q_{o_2} = *2-year allowable peak outflow = 17 cfs*

$$V_2 \quad \left| (35.1)(36.2) \quad \left(\frac{(35.1)(21)}{4} \right) \quad \left(\frac{(17)(36.2)}{2} \right) \quad \frac{3(17)(21)}{4} \right| 60$$

$$V_2 = 52{,}764 \text{ ft}^3 = 1.21 \text{ ac.ft.}$$

Repeat Steps 2 through 4 for the 10-year storm, as follows:

5. Determine the 10-year critical storm duration $T_{d_{10}}$, using **Equation 5-5** as follows:

$$T_{d_{10}} \quad \sqrt{\frac{2(.59)(25.0)(189.2)(22.1 \quad 21/4)}{24}} \quad 22.1$$

$$T_{d_{10}} \quad \sqrt{3918.6} \quad 22.1$$

$$T_{d_{10}} \quad 40.5 \ min.$$

Where:

$T_{d_{10}}$ = *10-year critical storm duration, min.*
C = *developed condition runoff coefficient = .59*
A = *drainage area = 25 acres*
t_c = *post-developed time of concentration = 21 min.*
$q_{o_{10}}$ = *24 cfs*
a_{10} = *189.2*
b_{10} = *22.1*

6. Solve for the 10-year critical storm duration intensity, I_{10}, using **Equation 5-2**, and the 10-year critical storm duration, $T_{d_{10}}$.

$$I_{10} \quad \frac{a}{b \quad T_{d_{10}}}$$

$$I_{10} \quad \frac{189.2}{22.1 \quad 40.5} \quad 3.02 \ in./hr.$$

7. Determine the 10-year peak inflow, $Q_{i_{10}}$, using the **Rational Equation** and the critical storm duration intensity I_{10}:

$$Q_{i_{10}} = C\, I_{10}\, A$$

Where: $Q_{i_{10}}$ = *10-year peak inflow*
C = *developed condition runoff coefficient* = .59
I_{10} = *critical storm intensity* = 3.02 *in./hr.*
A = *drainage area* = 25.0 *ac.*

$$Q_{i_{10}} = (.59)(3.02)(25.0)$$

$$Q_{i_{10}} = 44.5 \text{ cfs}$$

8. Determine the required 10-year storage volume for the 10-year critical storm duration, $T_{d_{10}}$, using **Equation 5-1**:

$$V_{10} \qquad \left[Q_{i_{10}} T_{d_{10}} \qquad \frac{Q_{i_{10}} t_c}{4} \qquad \frac{q_{o_{10}} T_{d_{10}}}{2} \qquad \frac{3 q_{o_{10}} t_c}{4} \right] 60$$

Where: V_{10} = *required storage, ft*^{3}
$Q_{i_{10}}$ = 44.5 *cfs*
C = .59
A = 25 *ac.*
$T_{d_{10}}$ = 40.5 *min.*
t_c = 21 *min.*
$q_{o_{10}}$ = 24 *cfs*

$$V_{10} \qquad \left[(44.5)(40.5) \qquad \frac{(44.5)(21)}{4} \qquad \frac{(24)(40.5)}{2} \qquad \frac{3(24)(21)}{4} \right] 60$$

$$V_{10} = 70{,}308 \text{ ft}^3 = 1.61 \text{ ac.ft.}$$

V_2 and V_{10} represent the total storage volume required for the 2-year and 10-year storms, respectively. **Table 5-6** provides a summary of the four different sizing procedures used in this chapter, as applied to **Example 1**. The engineer should choose one of these methods based on the complexity and size of the watershed and the chosen hydrologic method. Using the stage-storage curve, a multi-stage riser structure can then be designed to control the appropriate storms and, if required, the water quality volume.

TABLE 5 - 6
Summary of Results: Storage Volume Requirement Estimates, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration - Graphical Solution	*1.16 ac.ft.*	*1.56 ac.ft.*
Modified Rational Method - Critical Storm Duration - Direct Solution	*1.21 ac.ft.* T_d = 36.2 min.	*1.61 ac.ft.* T_d = 40.5 min.

5-5 STAGE-STORAGE CURVE

By using one of the above methods for determining the storage volume requirements, the engineer now has sufficient information to place and grade the proposed stormwater facility. Remember, **this is a preliminary sizing which needs to be refined during the actual design**. By trial and error, the approximate required volume can be achieved by designing the basin to fit the site geometry and topography. The storage volume can be computed by planimetering the contours and creating a ***stage-storage curve***.

5-5.1 Storage Volume Calculations

For retention/detention basins with vertical sides, such as tanks and vaults, the storage volume is simply the bottom surface area times the height. For basins with graded (2H:1V, 3H:1V, etc.) side slopes or an irregular shape, the stored volume can be computed by the following procedure. **Figure 5-10** represents the stage-storage computation worksheet completed for **Example 1**. A blank worksheet can be found at the end of this chapter (see **Figure 5-27**). (Note that other methods for computing basin volumes are available, such as the Conic Method for Reservoir Volumes, but they are not presented here.)

Procedure:

1. Planimeter or otherwise compute the area enclosed by each contour and enter the measured value into Columns 1 and 2 of **Figure 5-10**. The invert of the lowest control orifice represents zero storage. This will correspond to the bottom of the facility for extended-detention or detention facilities, or the permanent pool elevation for retention basins.

2. Convert the planimetered area (often in square inches) to units of square feet in Column 3 of **Figure 5-10**.

3. Calculate the average area between each contour.

   The average area between two contours is computed by adding the area planimetered for the first elevation, column 3, to the area planimetered for the second elevation, also Column 3, and then dividing their sum by 2. This average is then written in Column 4 of **Figure 5-10**.

   From **Figure 5-10**:

   Average area, elevation *81-82:* $\dfrac{0 + 1800}{2} = 900\ ft^2$.

   Average area, elevation *82-84*: $\dfrac{1800 + 3240}{2} = 2{,}520\ ft^2$.

   Average area, elevation *84-86:* $\dfrac{3240 + 5175}{2} = 4{,}207\ ft^2$.

   This procedure is repeated to calculate the average area found between any two consecutive contours.

4. Calculate the *volume* between each contour by multiplying the average area from step 3 (Column 4) by the contour interval and placing this product in Column 6. From **Figure 5-10**:

   Contour interval between *81* and $82 = 1\ ft. \times 900\ ft^2 = 900\ ft^3$
   Contour interval between *82* and $84 = 2\ ft. \times 2{,}520\ ft^2 = 5{,}040\ ft^3$

   This procedure is repeated for each measured contour interval.

FIGURE 5 - 10
Stage-Storage Computation Worksheet, Example 1

PROJECT: **EXAMPLE 1** SHEET ____ OF ____

COUNTY: ________________ COMPUTED BY: ____________ DATE: ________

DESCRIPTION: ____

ATTACH COPY OF TOPO: SCALE - 1" = **30** *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME *(ft³)*	TOTAL VOLUME *(ac.ft.)*
81	0	0				0	0
			900	1	900		
82	2.0	1800				900	.02
			2520	2	5040		
84	3.6	3240				5940	.14
			4207	2	8414		
86	5.75	5175				14354	.33
			7614	2	15228		
88	11.17	10053				29582	.68
			12991	2	25982		
90	17.7	15930				55564	1.28
			20700	2	41400		
92	28.3	25470				96964	2.23
			31102	1	31102		
93	40.8	36734				128066	2.94
			38105	1	38105		
94	43.9	39476				166171	3.81

5. Sum the volume for each contour interval in Column 7. Using **Figure 5-10,** this is simply the sum of the volumes computed in the previous step:

Contour *81, Volume = 0*

Contour *82, Volume =* $0 + 900 = 900\ ft^3$

Contour *84, Volume =* $900 + 5{,}040 = 5{,}940\ ft^3$

Contour *86, Volume =* $5{,}940 + 8{,}414 = 14{,}354\ ft^3$

Column 8 allows for the volume to be tabulated in units of acre-feet: $ft^3 \div 43{,}560\ ft^2/ac.$

This procedure is then repeated for each measured contour interval.

6. Plot the stage-storage curve with *stage* on the y-axis versus *storage* on the x-axis. **Figure 5-11** represents the stage-storage curve for **Example 1** in units of feet (stage) versus acre-feet (storage).

The stage-storage curve allows the designer to estimate the *design high water elevation* for each of the design storms if the required storage volume has been determined. This allows for a preliminary design of the riser orifice sizes and their configuration.

5-6 WATER QUALITY AND CHANNEL EROSION CONTROL VOLUME CALCULATIONS

Virginia's Stormwater Management Regulations require that the first flush of runoff, or the water quality volume, be treated to enhance water quality. The ***water quality volume*** (V_{wq}) is the first 0.5 inches of runoff from the impervious area of development. The water quality volume must be treated
using one or a combination of BMP's depending on the total size of the contributing watershed, amount of impervious area, and site conditions. (Refer to **Chapters 2** and **3** for **BMP Selection Criteria** and **BMP Minimum Standards and Specifications**, respectively.)

The water quality volume is calculated as follows:

$$V_{wq}\ (ft^3) = \text{Impervious area}\ (ft^2) \times (½\ in.) / (12\ in./ft.)$$

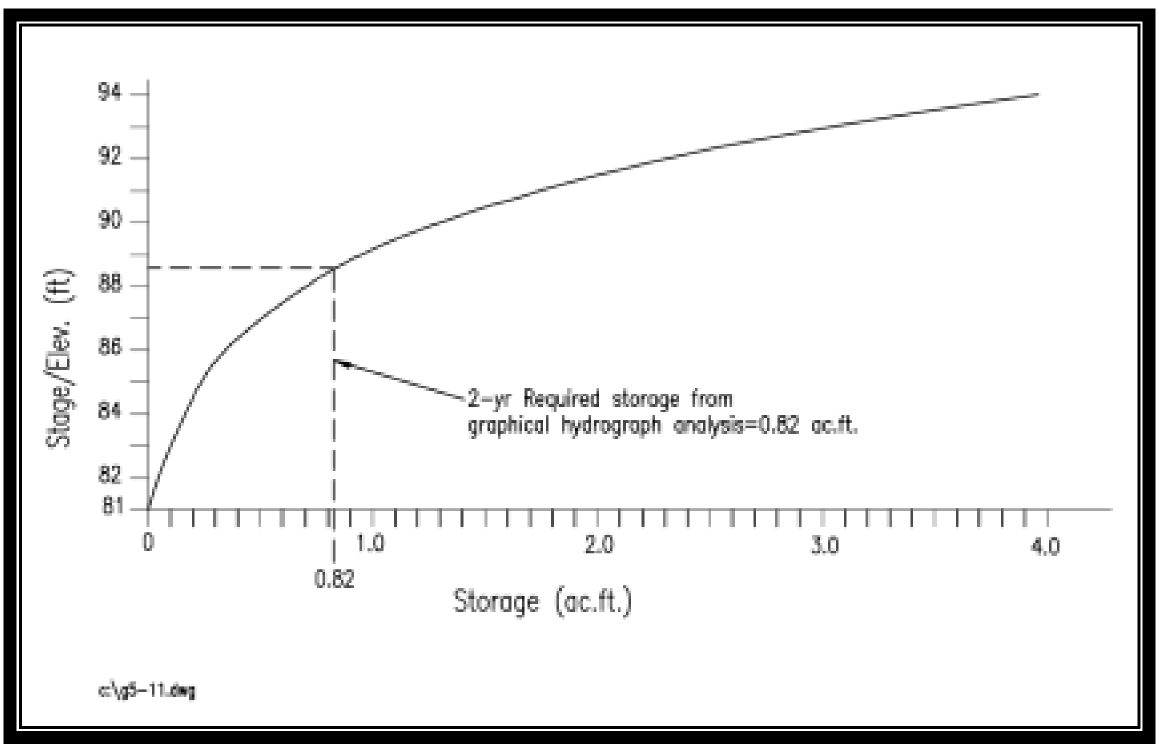
$$V_{wq}\ (ac.ft.) = V_{wq}\ (ft^3) / 43{,}560\ ft^2/ac.$$

The water quality volume for a wet BMP may be dependent on the specific design criteria for that BMP based on the watershed's imperviousness or the desired pollutant removal efficiency (using performance-based or technology-based criteria, respectively). The design criteria for each of the

BMPs, including extended-detention and retention basins, infiltration devices, constructed wetlands, marshes, etc., are presented in **Chapter 3**. This discussion is focused on the calculations associated with the control of the water quality volume in extended-detention and retention basins.

Virginia's Stormwater Management Regulations allow for the control of downstream channel erosion by detaining a specified volume of runoff for a period of time. Specifically, 24-hour extended detention of the runoff from the 1-year frequency storm is proposed as an alternate criteria to the 2-year peak rate reduction specified in MS-19 of the Virginia Erosion and Sediment Control Regulations, and the channel erosion component of the Virginia Stormwater Management Regulations. The channel erosion control volume (V_{ce}) is calculated by first determining the depth of runoff (in inches) based on the fraction of rainfall to runoff (runoff curve number) and then multiplying the runoff depth by the drainage area to be controlled. This procedure will be discussed in **5-6.3**.

FIGURE 5 - 11
Stage vs. Storage Curve, Example 1



5-6.1 Retention Basins - Water Quality Volume

The permanent pool feature of a retention basin allows for settling of particulate pollutants, such as sediment and other pollutants that attach adsorb to these particulates. Therefore, it is essential that the volume of the pool be both large enough and properly configured to prevent ***short-circuiting***. (Short-circuiting results when runoff enters the pool and exits without sufficient time for the settling process to occur.)

The permanent pool, or "dead" storage volume, of a retention facility is a function of the water quality volume. For example, a permanent pool sized to contain four times the water quality volume provides greater pollutant removal capacity than a permanent pool sized to contain two times the water quality volume. **Chapter 3** provides the pollutant removal efficiencies for various permanent pool sizes and criteria for permanent pool geometry.

Example 1 analyzes a 25-acre watershed. The water quality volume and permanent pool volume calculations for a retention basin serving this watershed are as follows:

Procedure

1. Calculate the water quality volume, V_{wq} , for the given watershed.

   From **Example 1**, the commercial/industrial development disturbs *11.9* acres, with *9.28* acres (*404,236 ft^2.*) of impervious cover after development.

   $$V_{wq} = 404{,}236\ ft^2 \times \tfrac{1}{2}\ in. / 12\ in./ft.$$

   $$= 16{,}843\ ft^3$$

   $$= 16{,}843\ ft^3 / 43{,}560\ ft^2/ac.$$

   $$V_{wq} = 0.38\ ac.ft.$$

2. Size the permanent pool based on the desired *pollutant removal efficiency* or the drainage area *impervious cover*.

   The pool volume will be sized based upon the desired pollutant removal efficiency. Referring to **Table 3.06-1**, the permanent pool must be sized for 4 x V_{wq} for a pollutant removal efficiency of 65%.

   $$\text{Permanent Pool Volume} = V_{wq} \times 4.0$$

   $$\text{Permanent Pool Volume} = 0.38\ ac.ft. \times 4.0 = 1.52\ ac.ft.$$

5-6.2 Extended-Detention Basins - Water Quality Volume and Orifice Design

A ***water quality extended-detention basin*** treats the water quality volume by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the water quality volume will allow the particulate pollutants to settle out of the *first flush* of runoff, functioning similarly to a permanent pool. Virginia's Stormwater Management Regulations pertaining to water quality specify a 30-hour draw down time for the water quality volume. This is a *brim draw down* time, beginning at the time of peak storage of the water quality volume. Brim-draw down time means the time required for the entire calculated volume to drain out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention orifice can be sized using either of the following methods:

1. Use the *maximum hydraulic head* associated with the water quality volume (V_{wq}) and calculate the orifice size needed to achieve the required draw down time, and route the the water quality volume through the basin to verify the actual storage volume used and the drawdown time.

2. Approximate the orifice size using the *average hydraulic head* associated with the water quality volume (V_{wq}) and the required draw down time.

The two methods for calculating the required size of the extended detention orifice allow for a quick and conservative design (Method 2 above) and a similarly quick estimation with a routing to verify the performance of the design (Method 1).

Method 1, which uses the *maximum hydraulic head* and maximum discharge in the calculation, results in a slightly larger orifice than the same procedure using the *average hydraulic head* (Method 2). The routing allows the designer to verify the performance of the calculated orifice size. As a result of the routing effect however, the actual basin storage volume used to achieve the draw down time will be less than the computed brim draw down volume. It should be noted that the routing of the extended detention of the runoff from storms larger than the water quality storm (such as the 1-year frequency storm for channel erosion control) will result in proportionately larger reduction in the actual storage volume needed to achieve the required extended detention. (Refer to **Section 5-6.3** for the extended detention design procedures for channel erosion protection.)

The procedure used to size an extended detention orifice includes the first steps of the design of a multi-stage riser for a basin controlling water quality and/or channel erosion, and peak discharge. These steps are repeated for sizing the 2-year and 10-year release openings. Other design storms may be used as required by ordinance or downstream conditions.

Method 1: Water quality orifice design using maximum hydraulic head and routing of the water quality volume.

A water quality extended-detention basin sized for two times the water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Procedure:

1. Calculate the water quality volume, V_{wq} , required for treatment.

   From **Example 1**:

   V_{wq} = 404,236 ft^2 x ½ in/ 12 in/ft = 16,843 ft^3

   V_{wq} = 16,843 ft^3/ 43,560 ft^2/ac = 0.38 ac.ft.

   For extended-detention basins, 2 x V_{wq} = 2(0.38 ac.ft.) = 0.76 ac.ft.= 33,106 ft^3.

2. Determine the maximum hydraulic head, h_{max} , corresponding to the required water quality volume.

   From the **Example 1** stage vs. storage curve (**Figure 5-11**):

   0.76 *ac.ft.* occurs at elevation 88 *ft*. (approximate). Therefore, h_{max} = 88 - 81 = 7.0 ft.

3. Determine the maximum discharge, Q_{max}, resulting from the 30-hour drawdown requirement.

   The *maximum discharge* is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge, and then multiplying by 2, to determine the maximum discharge.

   From **Example 1**:

$$Q_{avg} \quad \frac{33{,}106 \;\; ft^3}{(30\,hr.)(3{,}600\,sec./hr.)} \quad 0.30 \;\; cfs$$

$$Q_{max} = 2 \text{ x } 0.30\ cfs = 0.60\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

   Insert the values for Q_{max} and h_{max} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: *Q = discharge, cfs*
C = dimensionless coefficient = 0.6
a = area of the orifice, ft^2
g = gravitational acceleration, 32.2 ft/sec^2
h = head, ft.

From **Example 1**:

For orifice area:

$$a = \frac{0.6}{0.6\sqrt{(2)(32.2)(7.0)}}$$

For orifice diameter:

$$a = 0.047\ ft^2 = \pi r^2 = \pi d^2/4$$

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.047\ ft^2)}{\pi}}$$

d = orifice diameter = 0.245 ft = 2.94"

Use a 3-inch diameter water quality orifice.

Routing the water quality volume (V_{wq}) of 0.76 *ac.ft.*, occurring at elevation 88 feet, through a 3-inch water quality orifice will allow the designer to verify the draw down time, as well as the maximum elevation of 88 feet.

<u>Route the water quality volume</u>.

This calculation will give the engineer the *inflow-storage-outflow relationship* in order to verify the actual storage volume needed for the extended detention of the water quality volume. The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the water quality volume, as opposed to the brim drawdown described in Method 2. The routing procedure is simply a more accurate analysis of the storage volume used while water is flowing into and out of the basin. Therefore, the actual volume of the basin used will be less than the volume as defined by the regulation. This procedure will come in handy if the site to be developed is tight and the area needed for the stormwater basin must be "squeezed" as much as possible.

The routing effect of water entering and discharging from the basin simultaneously will also result in the actual drawdown time being less than the calculated 30 hours. Judgement should be used to determine whether the orifice size should be reduced to achieve the required 30 hours or if the actual time achieved will provide adequate pollutant removal.

NOTE: The designer will notice a significant reduction in the actual storage volume used when routing the extended detention of the runoff from the 1-year frequency storm (channel erosion control). Please refer to **Chapter 5-6.3** and **Chapter 5-11** for the appropriate design procedures when extended detention is provided for channel erosion control.

Routing the water quality volume depends on the ability to work backwards from the design runoff volume of 0.5 inches to find the rainfall amount. Using SCS methods, the rainfall needed to generate 0.5 inches of runoff from an impervious surface (*RCN=98*) is 0.7 inches. The SCS design storm is the Type II, 24-hour storm. Therefore, the *water quality storm* using SCS methods is defined as the SCS Type II, 24-hour storm, with a rainfall depth = 0.7 inches.

The rational method does not provide a design storm derived from a specified rainfall depth. Its rainfall depth depends on the storm duration (watershed t_c) and the storm return frequency. **Since the water quality storm varies with runoff amount, not the design storm return frequency, an input runoff hydrograph representing the water quality volume cannot be generated using rational method parameters**. Therefore Method 1, routing of the water quality volume, must use SCS methods. See **Chapter 4** for details on SCS methods.

Continuing with **Example 1**, the procedure is as follows:

Procedure (contd.)**:**

5. Calculate a stage-discharge relationship using the **Orifice Equation, Equation 5-6** and the orifice size determined in Step 4.

From **Example 1**, using the 3-inch diameter orifice, the calculation is as follows:

$$Q \quad Ca\sqrt{2gh}$$

Orifice Equation 5-6

$$Q \quad 0.6(.047)\sqrt{(2)(32.2)(h)}$$

$$Q \quad 0.22\sqrt{h}$$

where: h = *water surface elevation minus the orifice's centerline elevation*, in ft.*

Note: If the orifice size is small relative to the anticipated head, h, values of h may be defined as the water surface elevation minus the invert of the orifice elevation.

7. Complete a stage-discharge table for the range of elevations in the basin, as shown in **Table 5-7**:

TABLE 5 - 7

Stage-Discharge Table: Water Quality Orifice Design

Elevation	*h* (*ft*)	*Q* (*cfs*)
81	0	0
82	1	0.2
83	2	0.3
84	3	0.4
85	4	0.4
86	5	0.5
87	6	0.5
88	7	0.6

8. Determine the time of concentration as defined in **Chapter 4** for the impervious area.

   From **Example 1**, the developed time of concentration, t_c = *0.46 hours*. The impervious area time of concentration, $t_{c_{imp}}$ = 0.09 hours, or 5.4 minutes.

9. Using $t_{c_{imp}}$, the stage-discharge relationship, the stage-storage relationship, and the impervious acreage (*RCN = 98*), route the water quality storm through the basin. The water quality storm for this calculation is the SCS Type 2, 24-hour storm, rainfall depth = 0.7 inches. (Note that the rainfall depth is established as the amount of rainfall required to generate 0.5 inches of runoff from the impervious area.)

   The water quality volume may be routed using a variety of computer programs such as TR-20, HEC-1, or other storage indication routing programs. Alternatively, it can be routed by hand using the storage indication routing procedure outlined in **Section 5-9** of this chapter.

10. Evaluate the discharge hydrograph to verify that the drawdown time from maximum storage to zero discharge is at least 30 hours. (Note that the maximum storage corresponds to the maximum rate of discharge on the discharge hydrograph.)

    The routing of the water quality volume using TR-20 results in a maximum storage elevation is 85.69 ft. versus the approximated 88.0 ft. The brim drawdown time is 17.5 hours (peak discharge occurs at 12.5 hours and .01 discharge occurs at 30 hours). For this example, the orifice size may be reduced to provide a more reasonable drawdown time and another routing performed to find the new water quality volume elevation.

METHOD 2: Water quality orifice design using *average hydraulic head* and *average discharge*.

The procedure described in Method 2 is presented in the next section. For the previous example, Method 2 results in a 2.5 inch orifice (versus a 3.0 inch orifice), and the design extended detention water surface elevation is set at 88 ft.(versus 85.69ft.). (It should be noted that trial two of Method 1 as noted above may result in a design water surface elevation closer to 88 ft.) If the basin is to control additional storms, such as the 2-year and/or 10-year storms, the additional storage volume would be "stacked" just above the water quality volume. The invert for the 2-year control, for example, would be set at 88.1 *ft.*

5-6.3 Extended-Detention Basins - Channel Erosion Control Volume and Orifice Design

Extended detention of a specified volume of stormwater runoff can also be incorporated into a basin design to protect downstream channels from erosion. Virginia’s Stormwater Management Regulations recommend 24-hour extended detention of the runoff from the 1-year frequency storm as an alternative to the 2-year peak rate reduction required by MS-19 of the Virginia Erosion and Sediment Control Regulations. A full discussion of this channel erosion criteria will be presented in a future Technical Bulletin, along with practical guidance from DCR on the effective implementation of the criteria. The discussion presented here is for the design of a channel erosion control extended-detention orifice.

The design of a channel erosion control extended-detention orifice is similar to the design of the water quality orifice in that two methods can be employed:

1. Use the *maximum hydraulic head* associated with the specified channel erosion control (V_{ce}) storage volume and calculate the orifice size needed to achieve the required draw down time and route the 1-year storm through the basin to verify the storage volume and the draw down time, or

2. Approximate the orifice size using the *average hydraulic head* associated with the channel erosion control volume (V_{ce}) and draw down time.

The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the channel erosion control volume (V_{ce}). The routing procedure simply provides a more accurate accounting of the storage volume used while water is flowing into and out of the basin, and results in less storage volume being used than the calculated brim storage volume associated with the maximum hydraulic head. The actual storage volume needed for extended detention of the runoff generated by the 1-year frequency storm will be approximately 60 percent of the calculated volume (V_{ce}) of runoff for curve numbers between 75 and 95 and time of concentration between 0.1 and 1 hour.

The following procedure illustrates the design of the extended-detention orifice for channel erosion control. Refer to **Chapter 6** for **Example 6.2** which includes the design of an extended-detention orifice for channel erosion control, Method 1, within the design of a multi-stage riser.

Method 2:

Procedure

1. Calculate the channel erosion control volume, V_{ce}.

   Determine the rainfall amount (inches) of the 1-year frequency storm for the local area where the project is located (**Appendix 4B**). With the rainfall amount and the runoff curve number (RCN), determine the corresponding runoff depth using the runoff Equation (**Chapter 4: Hydrologic Methods - SCS** TR-55) or the Rainfall - Runoff Depth Charts (**Appendix 4C**).

   From **Example 2**:

   1-year rainfall = 2.7 inches, RCN = 75; using **Appendix 4C**, the 1-year frequency depth of runoff = 0.8 inches, therefore:

   $$V_{ce} = 25\ ac. \text{ x } 0.8\ in. \times 1'/12'' = 1.66\ ac.ft.$$

   To account for the routing effect, reduce the channel erosion control volume:

   $$V_{ce} = (0.6)(1.66\ ac.ft.) = 1.0\ ac.ft. = 43{,}560\ ft.^3$$

2. Determine the *average hydraulic head*, h_{avg}, corresponding to the required channel erosion control volume.

   From **Example 2** - Stage - Storage Curve: 1.0 *ac.ft.* occurs at elevation 89.0 *ft.* Therefore,

   $$h_{avg} = (89 - 81) / 2 = 4.0\ ft.$$

3. Determine the *average discharge*, Q_{avg}, resulting from the 24-hour draw down requirement.

The average discharge is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge.

From **Example 2**:

$$Q_{avg} \quad \frac{43{,}560\ ft^3}{(24\,hr.)(3{,}600\,sec./hr.)} \quad 0.5\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

Insert the values for Q_{avg} and h_{avg} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q \quad Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a \quad \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: Q = *discharge, cfs*
C = *dimensionless coefficient = 0.6*
a = *area of the orifice,* ft^2
g = *gravitational acceleration, 32.2* ft/sec^2
h = *head, ft.*

From **Example 2**:

For orifice area:

$$a \quad \frac{0.5}{0.6\sqrt{(2)(32.2)(4.0)}}$$

$$a \quad 0.052\ ft^2 \quad \text{´}\,r^2 \quad \text{´}\,d^2/4$$

For orifice diameter:

$$d \quad \sqrt{\frac{4a}{\text{´}}} \quad \sqrt{\frac{4(0.052\ ft^2)}{\text{´}}}$$

d = orifice diameter = 0.257 ft = 3.09 inches
Use 3.0-inch diameter channel erosion extended detention orifice

The use of Method 1, utilizing the maximum hydraulic head and a routing of the 1-year storm is illustrated in **Chapter 6: Example 6.2**. Methd 1 results in a 3.7" diameter orifice and a routed water surface elevation of 88.69 ft. Additional storms to may be "stacked" just above this volume if additional controls are desired.

5-7 MULTI-STAGE RISER DESIGN

A principal spillway system that controls the rate of discharge from a stormwater facility will often use a multi-stage riser for the drop inlet structure.

A multi-stage riser is a structure that incorporates separate openings or devices at different elevations to control the rate of discharge from a stormwater basin during multiple design storms. Permanent multi-stage risers are typically constructed of concrete to help increase their life expectancy; they can be precast or cast-in-place. The geometry of risers will vary from basin to basin. The engineer can be creative to provide the most economical and hydraulically efficient riser design possible. **Figure 3-02.1** in **Chapter 3** provides some examples of multi-stage riser structures.

In a stormwater management basin design, the multi-stage riser is of utmost importance since it controls the design water surface elevations. In designing the multi-stage riser, many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. Each iterative routing requires that the facility's size (*stage-storage* curve) and outlet shape (*stage-discharge table* or *rating curve*) be designed and tested for performance. Prior to final design, it is helpful to approximate the required storage volume and outlet shape using one of the "shortcut" methods, as described in **Section 5-4**. In doing this, the number of iterations may be reduced. The following procedures outline methods for approximating and then completing the design of a riser structure. (These design procedures are illustrated in the examples found in **Chapter 6**.)

<u>Information needed</u>:

1. The hydrology for the watershed or drainage area to be controlled, calculated by using one of the methods outlined in **Chapter 4**, and

2. The allowable release rates for the facility, as established by ordinance or downstream conditions.

The design procedure provided here will incorporate the traditional 2-year and 10-year design storms and the pre-developed hydrology will establish the allowable discharge rates of the developed watershed. It should be noted that any design storm, 1-year, 5-year, etc., can be substituted into this design procedure, as required.

Procedure:

STEP 1 Determine Water Quality or Extended Detention Requirements

Calculate the water quality volume and decide what method (extended-detention or retention) will be used to treat it, and/or calculate the channel erosion control volume for extended-detention, if required. (Virginia's Stormwater Management Regulations state that the **water quality volume is equal to the first 0.5 inch of runoff multiplied by the total impervious area of the land development project, and that the channel erosion control volume for extended detention is the runoff generated by the site during the 1-year frequency storm**.)

a. **Water Quality Extended-Detention Basin**: The water quality volume must be detained and released over 30-hours. The established pollutant removal efficiency is based on a 30-hour drawdown.

b. **Water Quality Retention Basin**: The volume of the permanent pool is established by the site impervious cover or the desired pollutant removal efficiency.

c. **Channel Erosion Control Extended-Detention Basin**: The channel erosion control volume must be detained and released over 24 hours.

Refer to **Chapter 3** for minimum BMP design standards and details.

STEP 2 Compute Allowable Release Rates

Compute the pre- and post-developed hydrology for the watershed. Sometimes, the pre-developed hydrology will establish the allowable release rate from the basin. Other times, the release rate will be established by downstream conditions. In either case, the post-developed hydrology will provide the peak inflow into the basin, as a peak rate (*cfs*) or a runoff hydrograph. Refer to **Section 5-3, Allowable Release Rates**.

STEP 3 Estimate the Required Storage Volume

Estimate the storage volume required using one of the "shortcut" volume estimate methods described in **Section 5-4**. The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for each of the appropriate design storms.

STEP 4 **Grade the Basin; Create Stage-Storage Curve**

After considering the site geometry and topography, select a location for the proposed stormwater management basin. By trial and error, size the basin such that it will hold the approximate required storage volume. Ensure that the storage volume is measured from the lowest stage outlet. (Note: the storage volume can be computed by planimetering the contours and creating a stage-storage relationship as described in **Section 5-5**.) Remember that this is a preliminary sizing which needs to be fine-tuned during the final design.

STEP 5a **Design Water Quality Orifice (Extended-Detention)**

The procedure for sizing the water quality orifice for an extended-detention basin is covered in **Section 5-6.2** of this chapter. Using either Method 1 or Method 2, the designer establishes the size of the water quality or stream channel erosion control orifice and the design maximum water surface elevation.

The lowest stage outlet of an extended-detention basin is the invert of the extended-detention (or water quality) orifice, which corresponds to zero storage. **Section 5-6.2** provides a detailed discussion for sizing the water quality orifice and **Chapter 6** gives examples of the calculation procedure.

STEP 5b **Set Permanent Pool Volume (Retention)**

In a retention pond, the permanent pool volume, from **STEP 1**, establishes the lowest stage outlet for the riser structure (not including a pond drain, if provided). The permanent pool elevation, therefore, corresponds to "0" storage for the design of the "dry" storage volume stacked on top of the permanent pool.

STEP 6 **Size 2-Year Control Orifice**

(The 2-year storm is used here to show the design procedure. Other design storms or release requirements can be substituted into the procedure.)

Knowing the 2-year storm storage requirement, from design **STEP 3**, and the water quality volume, from design **STEP 1**, the engineer can do a preliminary design for the 2-year release opening in the multi-stage riser. To complete the design, some iterations may be required to meet the allowable release rate performance criteria. This procedure is very similar to the water quality orifice sizing calculations:

1. Approximate the 2-year maximum head, $h_{2_{max}}$.

Establish the approximate elevation of the 2-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the water quality volume elevation from the approximate 2-year maximum water surface elevation to find the 2-year maximum head, $h_{2_{max}}$. If there are no water quality requirements, use the elevation of the basin bottom or invert.

2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$, from **STEP 2**.

3. Calculate the size of the 2-year control release orifice using the **Rearranged Orifice Equation, Equation 5-7** and solve for the area, *a*, in ft^2.

The engineer may choose to use any one of a variety of orifice shapes or geometries. Regardless of the selection, the orifice will initially act as a weir until the top of the orifice is submerged. Therefore, the discharges for the first stages of flow are calculated using the weir equation:

$$Q_w = C_w L h^{1.5}$$

Equation 5-8
Weir Equation

where:
Q_w = *weir flow discharge, cfs*
C_w = *dimensionless weir flow coefficient, typically equal to 3.1 for sharp crested weirs. Refer to **Table 5-8**.*
L = *length of weir crest, ft.*
h = *head, ft., measured from the water surface elevation to the crest of the weir*

Flow through the rectangular opening will transition from *weir flow* to *orifice flow* once the water surface has risen above the top of the opening. This orifice flow is expressed by the orifice equation. The area, *a*, of a rectangular orifice is written as $a = L \times H$,

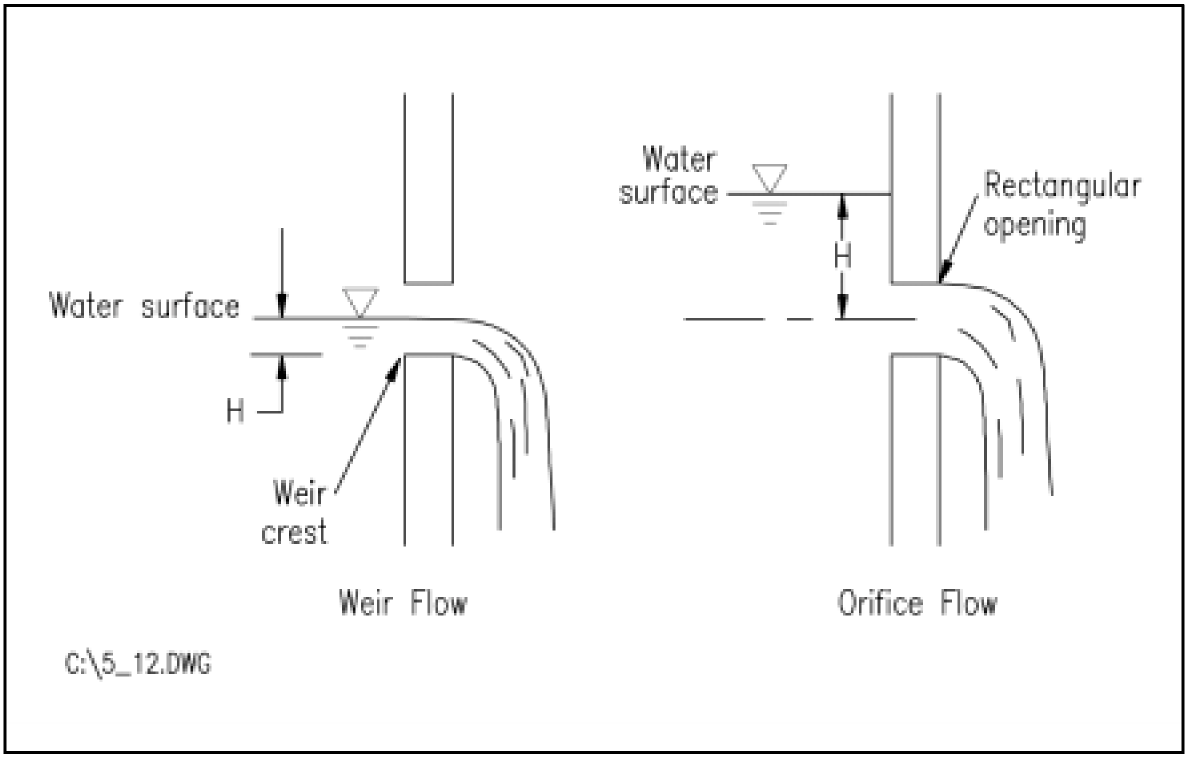
where:
L = *length of opening, ft.*
H = *height of opening, ft.*

Figure 5-12 shows a rectangular orifice acting as a weir at the lower stages and as an orifice after the water surface rises to height *H*, the height of the opening.

4. Develop the stage-storage-discharge relationship for the 2-year storm.

Calculate the discharge using the orifice equation and, if a rectangular opening is used, the weir equation as needed for each elevation specified on the stage-storage curve. Record the discharge on a Stage-Storage-Discharge Worksheet. **Figure 5-13** shows a completed Stage-Storage-Discharge Worksheet for **Example 2**. A blank worksheet is provided in **Appendix 5D**.

FIGURE 5 - 12
Weir and Orifice Flow



<u>STEP 7</u> **Check Performance of 2-Year Opening**

(Note: This step may not be necessary if the design is to be completed using one of the shortcut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

1. Check the performance of the 2-year control opening by a) *reservoir routing the 2-year storm through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in* ***Section 5-9*** *of this chapter*. **Verify that the 2-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening or provide additional storage and repeat **STEP 6**.

This procedure presents just one of many riser configurations. The engineer may choose to use any type of opening geometry for controlling the design storms and, with experience, may come to recognize the most efficient way to configure the riser. Note that if a weir is chosen for the 2-year storm control, the procedures outlined here for the 10-year storm may be used by substituting with the appropriate values for the 2-year storm. Refer to **Figure 3-02.1** for several different riser shapes.

STEP 8 **Size 10-Year Control Opening**

The design of the 10-year storm control opening is similar to the procedure used in sizing the 2-year control opening:

1. From the routing results, identify the exact 2-year water surface elevation.

2. Set the invert of the 10-year control just above the 2-year design water surface elevation and determine the corresponding storage volume from the stage-storage curve. Add this elevation, storage, and 2-year discharge to the stage-storage-discharge worksheet, **Figure 5-13**.

> ***The 10-year control invert may be set at a small distance, such as 0.1 feet minimum, above the 2-year maximum water surface elevation. If the 2-year orifice is also to be used for the 10-year control, the head is measured from the maximum water surface elevation to the centerline of the 2-year orifice. See Figure 5-14.***

3. Establish the approximate 10-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the invert elevation of the 10-year control (from Step 2 above) from the approximate 10-year maximum water surface elevation to find the 10-year maximum head, $h_{10_{max}}$.

4. Determine the maximum allowable 10-year discharge rate, $Q_{10_{allowable}}$, from **STEP 2**.

5. Calculate the required size of the 10-year release opening. The engineer may choose between a circular and rectangular orifice, or a weir. If a weir is chosen, the weir flow equation can be rearranged to solve for *L* as follows.

$$Q_W = C_W L h^{1.5}$$

Equation 5-8
Weir Equation

$$L = Q_{10_{allowable}} / C_W h^{1.5}$$

Equation 5-9
Rearranged Weir Equation

Where:

L = *length of weir required, ft.*
C_W = *dimensionless weir flow coefficient, see* ***Table 5-8***
$Q_{10_{allowable}}$ = *10-year allowable riser weir discharge, cfs*
h = *hydraulic head; water surface elevation minus the weir crest elevation*

6. Develop the stage-storage-discharge relationship for the 10-year storm. Calculate the discharge for each elevation specified on the stage-storage curve, and record the discharge on a Stage-Storage-Discharge Worksheet, as shown in **Figure 5-13**.

Any weir length lost to the trash rack or debris catcher must be accounted for. See Chapter 3 for Trash Rack Specifications and example riser configurations.

TABLE 5 - 8
Weir Flow Coefficients

WEIR FLOW COEFFICIENTS, C			
Measured head, *h*, (*ft.*)	**Breadth of weir crest (*ft.*)**		
	0.50	**0.75**	**1.00**
0.2	2.80	2.75	2.69
0.4	2.92	2.80	2.72
0.6	3.08	2.89	2.75
0.8	3.30	3.04	2.85
1.0	3.32	3.14	2.98
1.2	3.32	3.20	3.08
1.4	3.32	3.26	3.20
1.6	3.32	3.29	3.28
1.8	3.32	3.32	3.31
2.0	3.32	3.32	3.30
3.0	3.32	3.32	3.32
4.0	3.32	3.32	3.32
5.0	3.32	3.32	3.32

Source: Kings Handbook of Hydraulics

<u>STEP 9</u> Check Performance of 10-Year Opening

(Note: This step may not be necessary if the design is to be completed using one of the short-cut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

Check the performance of the 10-year control opening by a) *reservoir routing the 2-year and 10-year storms through the basin using an acceptable reservoir routing computer program* (see **Appendix**) or by b) *doing the long hand calculations outlined in* ***Section 5-9***. **Verify that the 10-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening and/or provide additional storage and repeat **STEP 8**.

<u>STEP 10</u> Perform Hydraulic Analysis

At this point, several iterations may be required to calibrate and optimize the hydraulics of the riser and the riser and barrel system. Drop inlet spillways should be designed so that full flow is established in the outlet conduit and riser at the lowest head over the riser crest as is practical. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This requires the riser to have a larger cross-sectional area than the outlet conduit.

As the water passes over the rim of the riser, the riser acts as a weir (**Figure 5-15a**); this discharge is described as ***riser weir flow control***. However, when the water surface reaches a certain height over the rim of the riser, the riser will begin to act as a submerged orifice (**Figure 5-15b**); such discharge is called ***riser orifice flow control***. The engineer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place. (This transition usually occurs during high hydraulic head conditions, such as between the 10-*yr.* and 100-*yr.* design high water elevations.)

Note in **Figure 5-15a & b** that the riser crest controls the flow, not the barrel. Thus, either condition can be described as ***riser flow control***. **Figure 5-15c & d** illustrates ***barrel flow control***. Barrel flow control occurs when the barrel controls the flow at the upstream entrance to the barrel (*barrel inlet flow control*, **Figure 5-15c**), or along the barrel length (*barrel pipe flow control*, **Figure 5-15d**).

Barrel flow control conditions illustrated in **Figure 5-15c & d** are desirable because they reduce or even eliminate cavitation forces, or surging and vibration (as described above), in the riser and barrel system. Cavitation forces in the riser and barrel system can greatly reduce the design flow capacity of the system. Cavitation forces may also cause vibrations that can damage the riser (especially corrugated metal risers) and the connection between the riser and barrel. This connection may crack and lose its watertight seal. Additionally, if a concrete riser is excessively tall with a minimum amount of the riser secured in the embankment, the cavitation forces may cause the riser to rock on its foundation, risking possible structural failure.

The surging, vibrations, and other cavitation forces result when the riser is restricting flow to the barrel such that the riser is flowing full and the barrel is <u>not</u> flowing full. This condition occurs when the flow through the riser structure transitions from *riser weir flow control* to *riser orifice flow control* <u>before</u> the barrel controls. Therefore, the barrel and riser system should be designed so that as the storm continues and the hydraulic head on the riser increases, **the barrel controls the flow <u>before</u> the riser transitions from riser weir flow control to riser orifice flow control.** This can be accomplished by checking the flow rates for the riser weir, riser orifice, and barrel inlet and outlet flow control at each stage of discharge. **The lowest discharge for any given stage will be the controlling flow**.

The following procedures are for designing and checking riser and barrel system hydraulics.

a. Riser Flow Control

During the design of the control orifices and riser weir, the geometry of the riser is established. Subsequently, the riser must be checked to determine at what stage it transitions from *riser weir* to *riser orifice* flow control. The riser weir controls the flow initially, and then as the water rises, the top of the riser acts as a submerged horizontal orifice. Thus, the flow transitions from riser weir flow control to riser orifice flow control as the water in the basin rises. The flow capacity of the riser weir is determined using the **Weir Equation**, **Equation 5-8**, and the flow capacity of the riser orifice is determined using the **Orifice Equation**, **Equation 5-6**, for each elevation. **The smaller of the two flows for any given elevation is the controlling flow**.

1. Calculate the flow, in *cfs,* over the riser weir using the standard **Weir Equation**, **Equation 5-8**, for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***weir length***, *L,* is the circumference or length of the riser structure, measured at the crest, less any support posts or trash rack. The ***head*** is measured from the water surface elevation to the crest of the riser structure (refer to **Figure 5-14**).

2. Calculate the flow, in *cfs,* through the riser structure using the standard **Orifice Equation**, **Equation 5-6,** for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***Orifice flow area***, *a,* is measured from the inside dimensions of the riser structure. The ***head*** is measured from the water surface elevation to the elevation of the orifice centerline, or, since the orifice is horizontal, to the elevation of the riser crest.

3. Compare the riser weir flow discharges to the riser orifice flow discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

b. Barrel Flow Control

Two types of barrel flow exist: 1) ***barrel flow with inlet control***, as shown in **Figure 5-15c**, and 2) ***barrel flow with outlet, or pipe flow control***, as shown in **Figure 5-15d**. For both types, different factors and formulas are used to compute the hydraulic capacity of the barrel. During barrel inlet flow control, the diameter of the barrel, amount of head acting on the barrel, and the barrel entrance shape play a part in controlling the flow. For barrel outlet, or pipe flow, control, consideration is given to the length, slope, and roughness of the barrel, and the elevation of the tailwater, if any, in the outlet channel.

1. Barrel Inlet Flow Control

Barrel inlet flow control means that the capacity of the barrel is controlled at the barrel entrance by the depth of headwater and the barrel entrance, which is acting as a submerged orifice. The flow through the barrel entrance can be calculated using the **Orifice Equation, Equation 5-6,** or by simply using the Pipe Flow Nomograph shown in **Figure 5-16.** This nomograph provides stage-discharge relationships for concrete culverts of various sizes. [Additional nomographs for other pipe materials and geometrics are available; refer to the U.S. Bureau of Public Roads (BPR) Hydraulic Engineering Circular (H.E.C.) 5.] The ***headwater***, or depth of ponding, is the vertical distance measured from the water surface elevation to the invert at the entrance to the barrel. Refer to **Figure 5-16** for ratios of *headwater* to *pipe diameter*, or *HW/D*. This nomograph, based on the orifice equation, provides flow rates for three possible hydraulic entrance shapes, as shown in **Figure 5-17**. During barrel inlet flow control, neither the barrel's length nor its outlet conditions are factors in determining the barrel's capacity. Note that when the *HW/D* design values exceed the chart values, the designer may use the orifice equation (**Equation 5-6**) to solve for the flow rate.

The inlet control nomographs are not truly representative of barrel inlet flow. These nomographs should be used carefully and with the understanding that they were developed to predict flow through highway culverts operating under inlet control. However, depending on the size relationship between the riser and outlet conduit, the inlet control nomograph may provide a

The following procedure outlines the steps to calculate the discharge during ***barrel inlet flow*** control conditions:

1. Determine the *entrance condition* of the barrel (see **Figure 5-17**).

2. Determine the *headwater to pipe diameter ratio* (*HW/D*) for each elevation specified on the stage-storage-discharge worksheet. *Headwater* is measured from the water surface elevation to the upstream invert of the barrel (see **Figures 5-14 and 5-18**).

3. Determine the *discharge*, *Q*, in *cfs*, using the inlet control nomograph for circular concrete pipe presented in **Figure 5-16** (or the BPR H.E.C. 5 pipe flow nomographs for other pipe materials), or the **Orifice Equation**, **Equation 5-6** (for *HW/D* values which exceed the range of the nomographs) for each elevation specified on the Stage-Storage-Discharge Worksheet. Enter the values on the worksheet.

2. Barrel Outlet Flow Control

Barrels flowing under outlet or pipe flow control experience full flow for all or part of the barrel length, as shown in **Figure 5-15d**.

The general pipe flow equation is derived by using the Bernouli and Continuity Principles and is simplified to:

$$Q = a\sqrt{\frac{2gh}{1 + K_m + K_p L}}$$

Equation 5 - 10
Pipe Flow Control Equation

Where:

Q = *discharge, cfs*
a = *flow area of the barrel,* ft^2
g = *acceleration due to gravity,* $ft./sec^2$
h = *elevation head differential, ft., see* ***Figure 5-18***
K_m = *coefficient of minor losses:* $K_e + K_b$
K_e = *entrance loss coefficient, see* ***Table 5-9***
K_b = *bend loss coefficient, typically = 0.5 for riser and barrel system*
K_p = *coefficient of pipe friction, see* ***Table 5-10***
l = *length of the barrel, ft.*

This equation is derived and further explained in the SCS's Engineering Field Manual, Chapter 3.

The following procedure outlines the steps to check for ***barrel outlet control***:

1. Determine the discharge for each elevation specified in the stage-storage-discharge table using the general **Pipe Flow Equation**, **Equation 5-10**.

2. Record the discharge on the stage-storage-discharge worksheet, **Figure 5-13**.

3. Compare the barrel inlet flow control discharges with the barrel outlet flow control discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

STEP 11 Size 100-Year Release Opening or Emergency Spillway

It is recommended that all stormwater impoundment structures have a vegetated emergency spillway, if possible. This provides a degree of safety to prevent overtopping of the embankment if the principal spillway should become clogged, or otherwise inoperative. **If an emergency spillway is not practical due to site constraints, the 100-year storm must be routed through the riser and barrel system**.

100-Year Release Opening

The design procedure for sizing the 100-year release opening is the same as that of the 10-year design, except that the 100-year storm values are used instead of the 10-year values.

Emergency Spillway

Refer to **Minimum Standard 3.03, Vegetated Emergency Spillway** in **Chapter 3** for location and design requirements of an emergency spillway and to **Section 5-8** in this chapter for the design procedure. An emergency spillway is a broad crested weir. It can act as a control structure by restricting the release of flow, or it can be used to safely pass the 100-year storm flow with a minimum of storage. The impact of the 100-year storm on the required storage is lessened by using an emergency spillway due to the spillway's ability to pass significant volumes of flow with little head. If an emergency spillway is not used, additional storage may be needed since the riser and barrel will usually pass only a small portion of the 100-year inflow. This remains true unless the riser and barrel are sized for the 100-year storm, in which case they will be oversized for the 2- and 10-year storms.

The following procedure can be used to design an emergency spillway that will safely pass, or control, the rate of discharge from the 100-year storm.

1. Identify the 10-year maximum water surface elevation based on the routing from **STEP 9**. This elevation will be used to establish the elevation of the 100-year release structure.

2. Determine the storage volume that corresponds to the 100-year control elevation from the stage-storage curve. Add this elevation, storage, and appropriate storm discharges to the Stage-Storage-Discharge Worksheet.

3. Set the invert of the emergency spillway at the 10-year high water elevation.

4. Determine the 100-year developed inflow from the hydrology.

> ***A distance of 0.1 feet, minimum, is recommended between the 10-year high water mark and the invert of the emergency spillway.***

5. Using the design procedure provided in **Chapter 5-8**, determine the required bottom width of the spillway, the length of the spillway level section, and the depth of flow through the spillway that adequately passes the 100-year storm within the available free board. The minimum free board required is 1 foot from the 100-year water surface elevation to the settled top of embankment.

6. Develop the stage-storage-discharge relationship for the 100-year storm. Calculate the

discharge for each elevation specified on the stage-storage curve and record the discharge on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. If a release rate is specified, then the TR-55 shortcut method can be used to calculate the approximate storage volume requirement. If a fixed storage volume is available, the same shortcut method can be used to decide what the discharge must be to ensure that the available storage is not exceeded. Refer to TR-55.

STEP 12 Calculate Total Discharge and Check Performance of 100-Year Control Opening

1. Calculate total discharge.

   The stage-storage-discharge table is now complete and the total discharge from the riser and barrel system and emergency spillway can be determined. The designer should verify that the barrel flow controls before the riser transitions from riser weir flow control to riser orifice flow control.

   The combined flows from the water quality orifice, the 2-year opening, the 10-year opening, and the riser will, at some point, exceed the capacity of the barrel. At this water surface elevation and discharge, the system transitions from riser flow control to barrel flow control. The total discharge for each elevation is simply the sum of the flows through the control orifices of the riser, or the controlling flow through the barrel and riser, whichever is **less**.

   In **Chapter 6,** the examples contain completed Stage-Storage-Discharge Worksheets. Notice that the flows that do not control are crossed out. The controlling flows are then summed in the total flow column to provide the total stage-storage-discharge relationship of the basin.

2. Check the performance of the 100-year control by a) *reservoir routing the 2-year, 10-year, and 100-year storms through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in* ***Section 5-9****.* **Verify that the design storm release rates are less than or equal to the allowable release rates, and that the 100-year design high water is:**

   a. at least 2 *ft.* lower than the settled top of embankment elevation if an emergency spillway is NOT used, or

   b. at least 1 *ft.* lower than the settled top of embankment if an emergency spillway is used.

Also, the designer should verify that the release rates for each design storm are not too low, which would result in more storage being provided than is required.

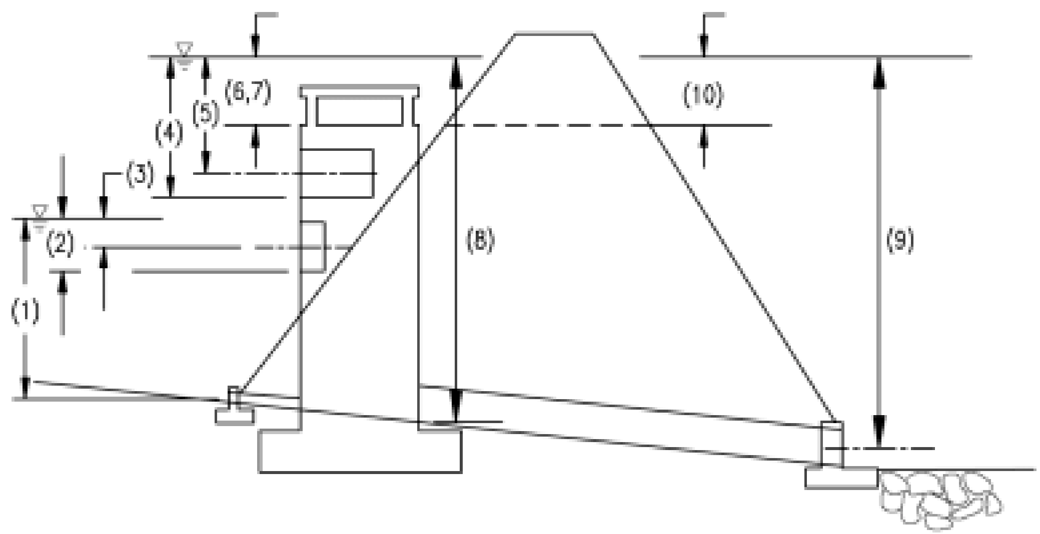
FIGURE 5 - 13
Stage - Storage - Discharge Worksheet, Example 1

ELEV (MSL)	STORAGE (ac-ft)	1-yr ext det ~~WATER QUALITY~~ ORIFICE (1)		TRIAL 2 10-YEAR CONTROL WEIR (4)		ORIFICE (5)		RISER STRUCTURE WEIR (6)		ORIFICE (7)		BARREL INLET (8)		OUTLET (9)		EMERGENCY SPILLWAY (10)		TOTAL Q (cfs)
		h	Q	h	Q	h	Q	h	Q	h	Q	HW/D	Q	h	Q	h	Q	
81	0	0	0															0
82	.02	1	0.3															0.3
84	.14	3	0.6															0.6
86	.33	5	0.7															0.7
88	.68	7	0.9															0.9
88.8	.90	7.8	0.98	0	0				0/0	0	0							1.0
89	.95	8	1.0	0.2	0.4				0/0.4	0.2	~~34.4~~	5.5	~~24~~	8.3	~~22.0~~			1.4
90	1.28	9	1.1	1.2	5.8				0/5.8	1.2	84.4	6.2	~~26~~	9.3	~~23.3~~			6.9
91	1.75	10	1.1	2.2	14.3				0/14.3	2.2	~~114.3~~	6.8	~~27~~	10.3	~~24.5~~			15.4
92	2.23	11	~~1.1~~	3.2	~~25.1~~				0/25.1	3.2	139.8	7.5	~~28~~	11.3	25.7			25.7
92.2	2.40	11.2	~~1.1~~	3.4	~~27.5~~			0	0/27.5	3.4	142.0	7.6	~~29~~	11.5	25.9	0	0	25.9
93	2.94	12.0	~~1.1~~	4.2	~~37.8~~			0.8	12.9/[illegible]	4.2	157.9	8.1	~~30~~	12.3	26.8	0.8	54	80.8
94	3.81	13.0	~~1.2~~	5.2	~~52.0~~			1.8	21.5/[illegible]	5.2	175.9	8.8	~~31~~	13.3	27.8	1.8	214	241.8

(6) Q for elevations 88.8 to 92.2 ft. represents 10-yr. weir flow (4)
Q for elevations 92.2 to 94 ft represents 10-yr weir flow plus the top of riser weir flow: $Q = 23.8 (h)^{1.5}$ where h = wse - 92.2 ft.

(7) $Q = 77.03 (h)^{1/2}$ where h = wse - 88.8 ft
(8) $Q = 8.5 (h)^{1/2}$ where h = wse - 80.75 ft
(9) $Q = 7.64 (h)^{1/2}$ where h = wse - 80.75 ft
(10) Figure 5-23: Design Data for Earth Spillways.

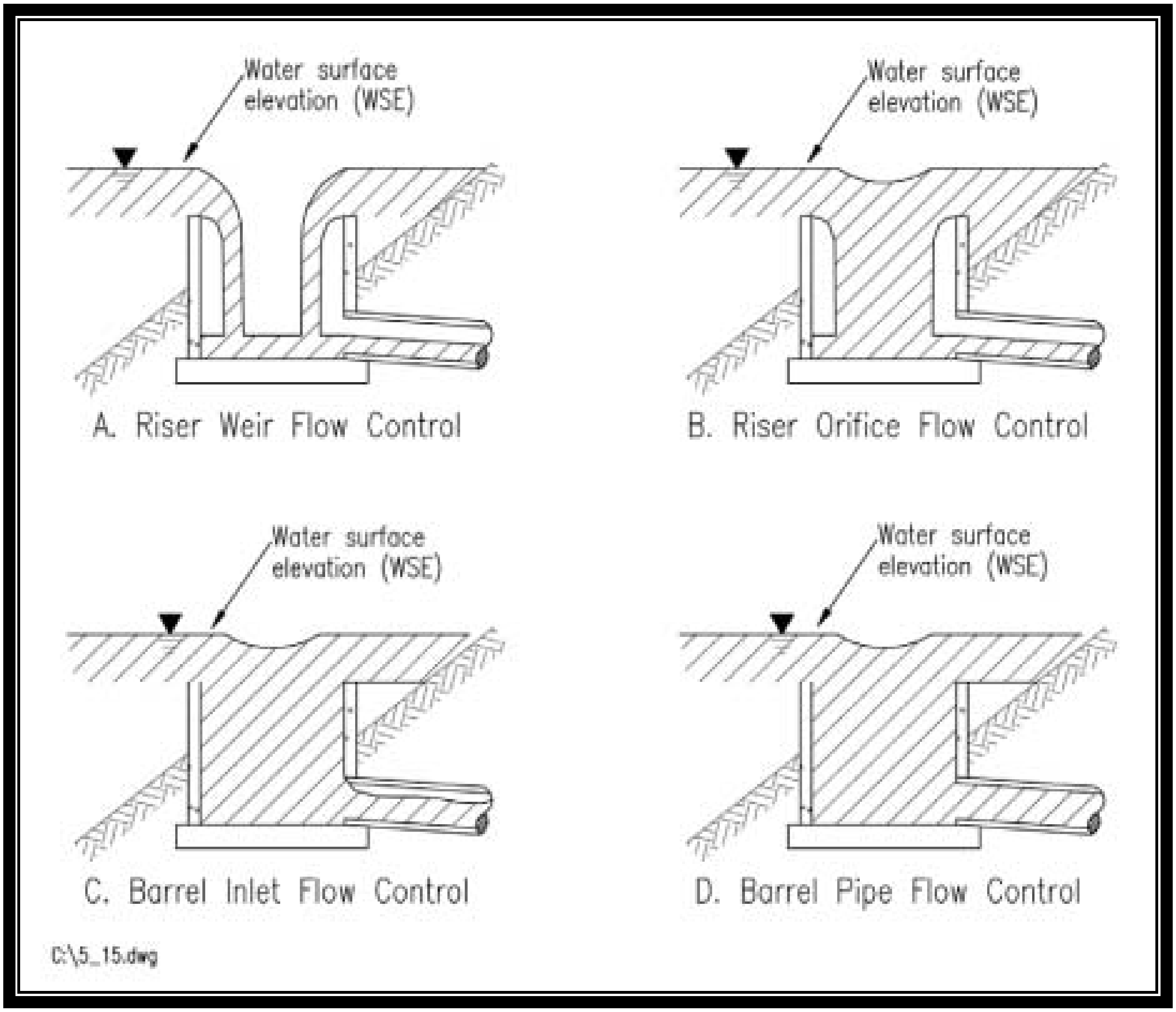
FIGURE 5 - 14
Typical Hydraulic Head Values - Multi-Stage Riser



1. WQ Orifice: H measured from water surface elevation (WSE) to centerline of pipe, or orifice. (orifice flow)
2. 2–year Control: Weir flow – H measured from WSE to invert of 2–year control weir.
3. 2–year Control: Orifice flow – H measured from WSE to centerline of opening (submerged).
4. 10–year Control: Weir flow – H measured from WSE to crest of opening.
5. 10–year Control: Orifice flow – H measured from WSE to centerline of opening.
6. Riser Structure: Weir flow – H measured from WSE to crest of riser top (if open).
7. Riser Structure: Orifice flow – H measured from WSE to crest of riser top, acting as horizontal orifice.
8. Barrel flow: Inlet control – H measured from WSE to upstream invert of outlet barrel.
9. Barrel flow: Outlet control – H measured from WSE to centerline of outlet of barrel or tailwater – whichever is higher.
10. Emergency Spillway: H measured from WSE to crest of Emergency Spillway.

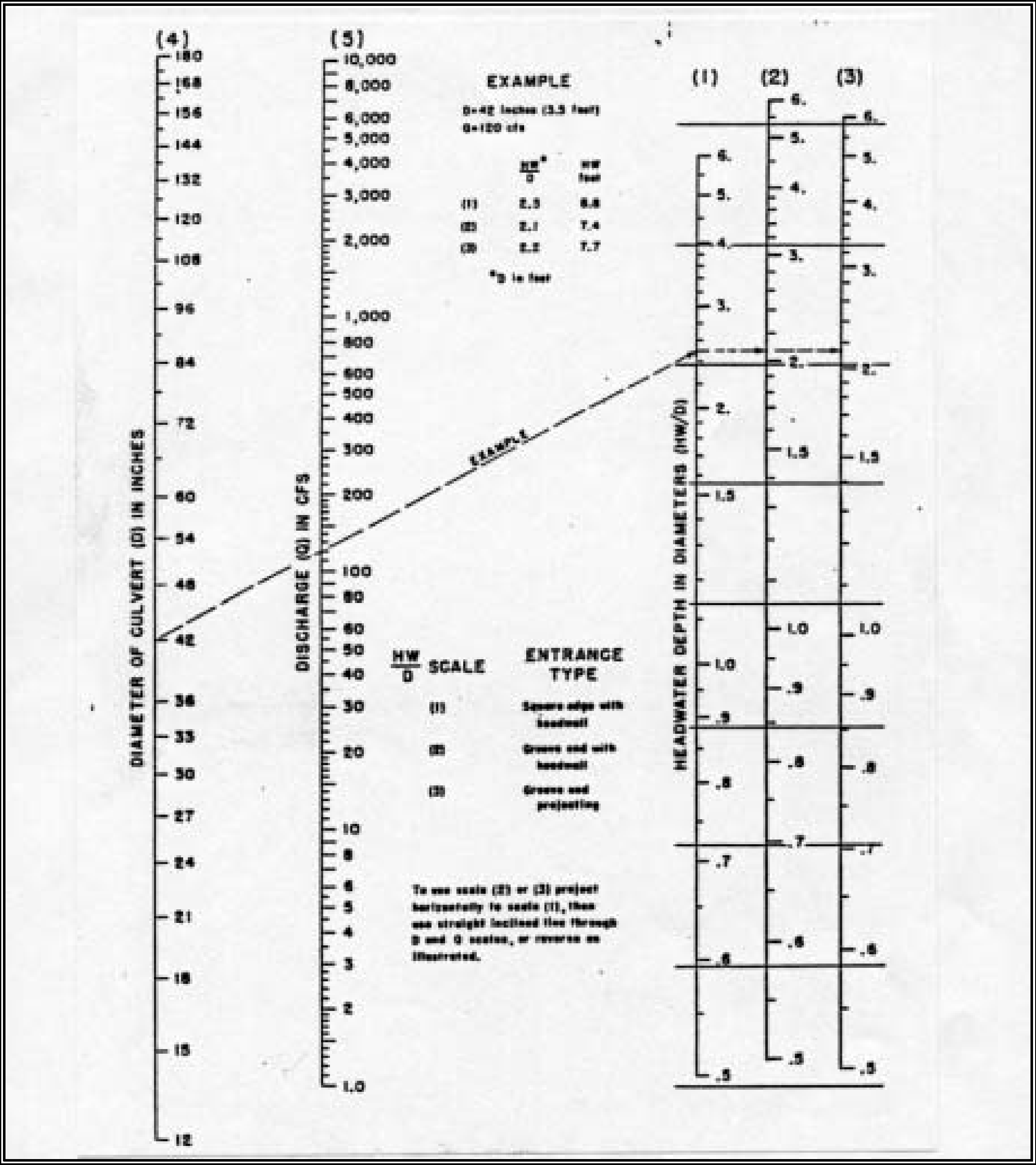
c:\5_14.dwg

FIGURE 5 - 15 a, b, c, & d
Riser Flow Diagrams



Source: SCS Engineering Field Manual **-** *Chapter 6*

FIGURE 5 - 16
Headwater Depth for Concrete Pipe Culverts With Inlet Control



Source: Bureau of Public Roads

FIGURE 5 - 17
Headwater Depth Entrance Conditions

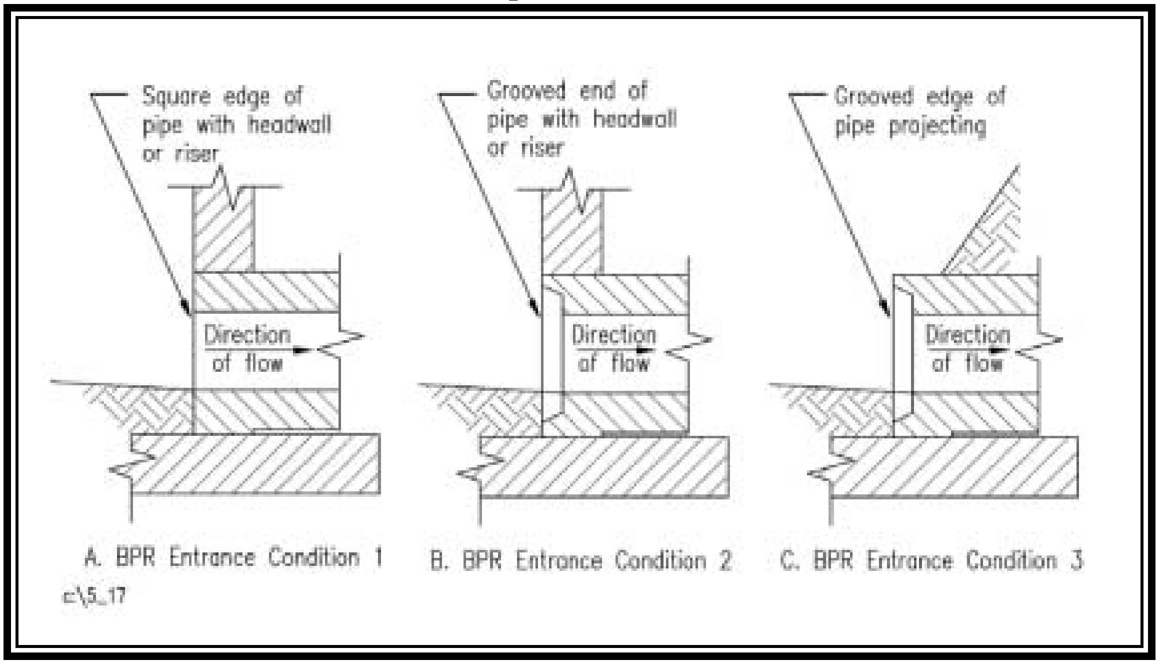


FIGURE 5 - 18
Hydraulic Head Values - Barrel Flow

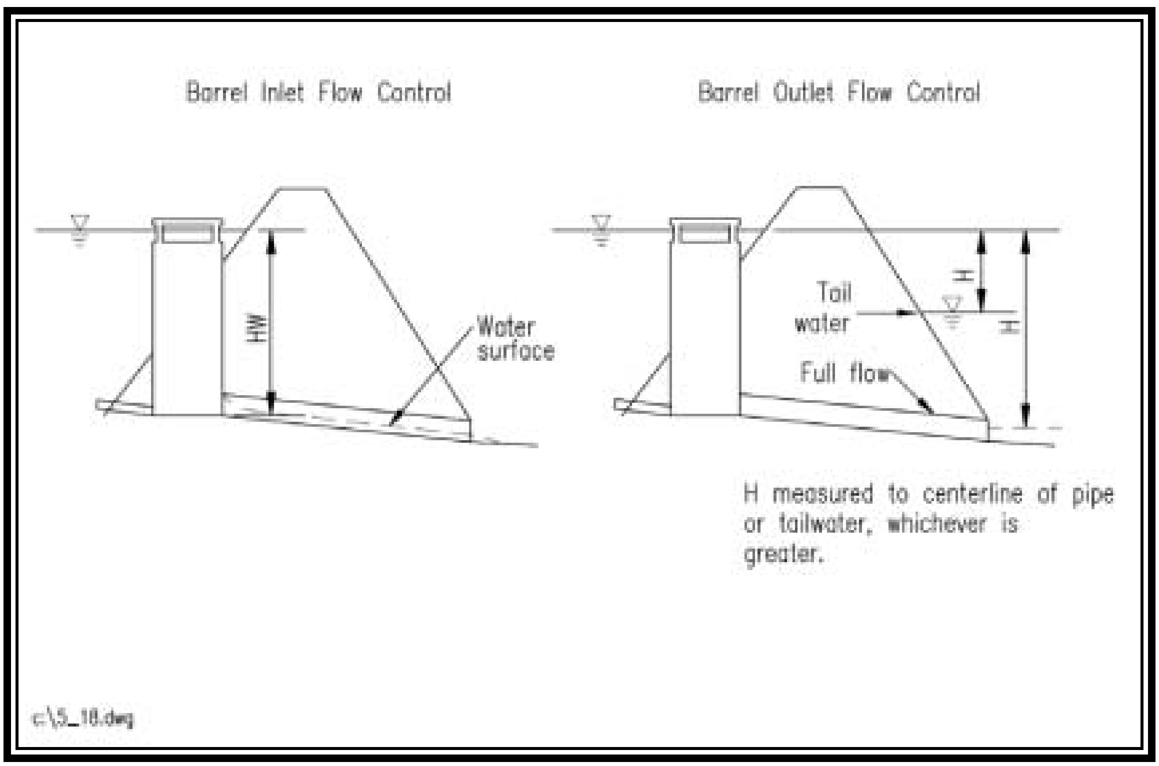


TABLE 5 - 9
Pipe Entrance Loss Coefficients - K_e

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square end	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square end	0.5
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5

**Note: "End-section conforming to fill slope" made of either metal or concrete, is the section commonly available from manufacturers. Based on limited hydraulic tests, it appears to be equivalent in operation to a headwall in either inlet or outlet control.*

Source - *Federal Highway Administration, Bureau of Public Roads*

TABLE 5 - 10

Head Loss Coefficients, K_p , for Circular and Square Conduits Flowing Full

HEAD LOSS COEFFICIENT, K_p, FOR CIRCULAR PIPE FLOWING FULL $K_p = \frac{5087 n^2}{d_i^{4/3}}$

Pipe diam. inches	Flow area sq. ft.	Manning's Coefficient of Roughness "n"															
		0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.151	0.168	0.187	0.206	0.226	0.247	0.269	0.292
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.140	.154	.168	.183	.199
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.136	.148
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0296	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.23	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.40	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.77	.01078	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.41	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.14	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.98	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.91	.00546	.00660	.00786	.00922	.01070	.01228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.07	.00428	.00518	.00616	.00723	.00839	.00963	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.62	.00348	.00421	.00501	.00588	.00683	.00784	.00892	.01007	.0113	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.57	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.90	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.63	.00217	.00262	.00312	.00366	.00424	.00487	.00554	.00626	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

HEAD LOSS COEFFICIENT, K_c, FOR SQUARE CONDUIT FLOWING FULL $K_c = \frac{29.16 n^2}{r^{4/3}}$

Conduit Size feet	Flow area sq. ft.	Manning's Coefficient of Roughness "n"				
		0.012	0.013	0.014	0.015	0.016
2×2	4.00	0.01059	0.01242	0.01440	0.01653	0.01880
2½×2½	6.25	0.00786	0.00922	0.01070	0.01228	0.01397
3×3	9.00	.00616	.00723	.00839	.00963	.01096
3½×3½	12.25	.00502	.00589	.00683	.00784	.00892
4×4	16.00	.00420	.00493	.00572	.00656	.00746
4½×4½	20.25	.00359	.00421	.00488	.00561	.00638
5×5	25.00	.00312	.00366	.00425	.00487	.00554
5½×5½	30.25	.00275	.00322	.00374	.00429	.00488
6×6	36.00	.00245	.00287	.00333	.00382	.00435
6½×6½	42.25	.00220	.00258	.00299	.00343	.00391
7×7	49.00	.00199	.00234	.00271	.00311	.00354
7½×7½	56.25	.00182	.00213	.00247	.00284	.00323
8×8	64.00	.00167	.00196	.00227	.00260	.00296
8½×8½	72.25	.00154	.00180	.00209	.00240	.00273
9×9	81.00	.00142	.00167	.00194	.00223	.00253
9½×9½	90.25	.00133	.00156	.00180	.00207	.00236
10×10	100.00	.00124	.00145	.00168	.00193	.00220

$$H_f = (K_p \text{ or } K_c)\, L\, \frac{v^2}{2g}$$

Nomenclature:

a = Cross-sectional area of flow in sq. ft.
d_i = Inside diameter of pipe in inches.
g = Acceleration of gravity = 32.2 ft per sec.
H_f = Loss of head in feet due to friction in length L.
K_c = Head loss coefficient for square conduit flowing full.
K_p = Head loss coefficient for circular pipe flowing full.
L = Length of conduit in feet.
n = Manning's coefficient of roughness.
Q = Discharge or capacity in cu. ft. per sec.
r = Hyd aulic radius in feet.
v = Mean velocity in ft. per sec.

Example 1: Compute the head loss in 300 ft of 24 in. diam. concrete pipe flowing full and discharging 30 c.f.s. Assume n = 0.015

$$v = \frac{Q}{a} = \frac{30}{3.14} = 9.55 \text{ f.p.s.};\quad \frac{v^2}{2g} = \frac{(9.55)^2}{64.4} = 1.42 \text{ ft}$$

$$H_f = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03 \text{ ft.}$$

Example 2: Compute the discharge of a 250 ft., 3×3 square conduit flowing full if the loss of head is determined to be 2.25 ft. Assume n = 0.014.

$$H_f = K_c L \frac{v^2}{2g};\quad \frac{v^2}{2g} = \frac{H_f}{K_c L} = \frac{2.25}{0.00839 \times 250} = 1.073 \text{ ft.}$$

$$v = \sqrt{64.4 \times 1.073} = 8.31;\quad Q = 9 \times 8.31 = 74.8 \text{ c.f.s.}$$

Head loss coefficients for circular and square conduits flowing full (Ref. NEH Section 5, ES-42)

<u>STEP 13</u> Design Outlet Protection

With the total discharge known for the full range of design storms, adequate outlet protection can now be designed. Protection is necessary to prevent scouring at the outlet and to help reduce the potential for downstream erosion by reducing the velocity and energy of the concentrated discharge. The most common form of outlet protection is a riprap-lined apron, constructed at zero grade for a specified distance, as determined by the outlet flow rate and tailwater elevation. The design procedure follows:

> ***Note that this procedure is for riprap outlet protection at the downstream end of an embankment conduit. It DOES NOT apply to continuous rock linings of channels or streams. Refer to Figure 5 -19.***

1. Determine the tailwater depth, for the appropriate design storm, immediately below the discharge pipe.

Typically, the discharge pipe from a stormwater management facility is sized to carry the allowable discharge from the 10-year frequency design storm. Manning's equation can be used to find the water surface elevation in the receiving channel for the 10-year storm, which represents the ***tailwater elevation***. If the tailwater depth is less than half the outlet pipe diameter, it is called a ***minimum tailwater condition***. If the tailwater depth is greater than half the outlet pipe diameter, it is called a ***maximum tailwater condition***. Stormwater basins that discharge onto flat areas with no defined channel may be assumed to have a *minimum tailwater condition.*

> ***Outflows from stormwater management facilities must be discharged to an adequate channel. Basins discharging onto a flat area with no defined channel will usually require a channel to be provided which can convey the design flows.***

2. Determine the required riprap size, D_{50}, and apron length, L_a.

Enter the appropriate figure, either **Figure 5-20: Minimum Tailwater Condition**, or **Figure 5-21: Maximum Tailwater Condition**, with the design discharge of the pipe spillway to read the required apron length, L_s. (The apron length should not be less than 10 feet.)

3. Determine the required riprap apron width, W.

When the pipe discharges directly into a well-defined channel, the apron shall extend across the channel bottom and up the channel banks to an elevation 1 foot above the maximum tailwater depth or the top of bank, whichever is less.

If the pipe discharges onto a flat area with no defined channel, the width of the apron shall be determined as follows:

a. The upstream end of the apron, next to the pipe, shall be 3 times wider than the diameter of the outlet pipe.

b. For a ***minimum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus the length of the apron.

c. For a ***maximum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus 0.4 times the length of the apron.

Using the same figure as in Step 2, above, determine the D_{50} riprap size and select the appropriate class of riprap, as shown in **Table 5-11**. Values falling between the table values should be rounded up to the next class size.

4. Determine the required depth of the rip rap blanket.

The depth of the rip rap blanket is approximated as: 2.25 x D_{50}

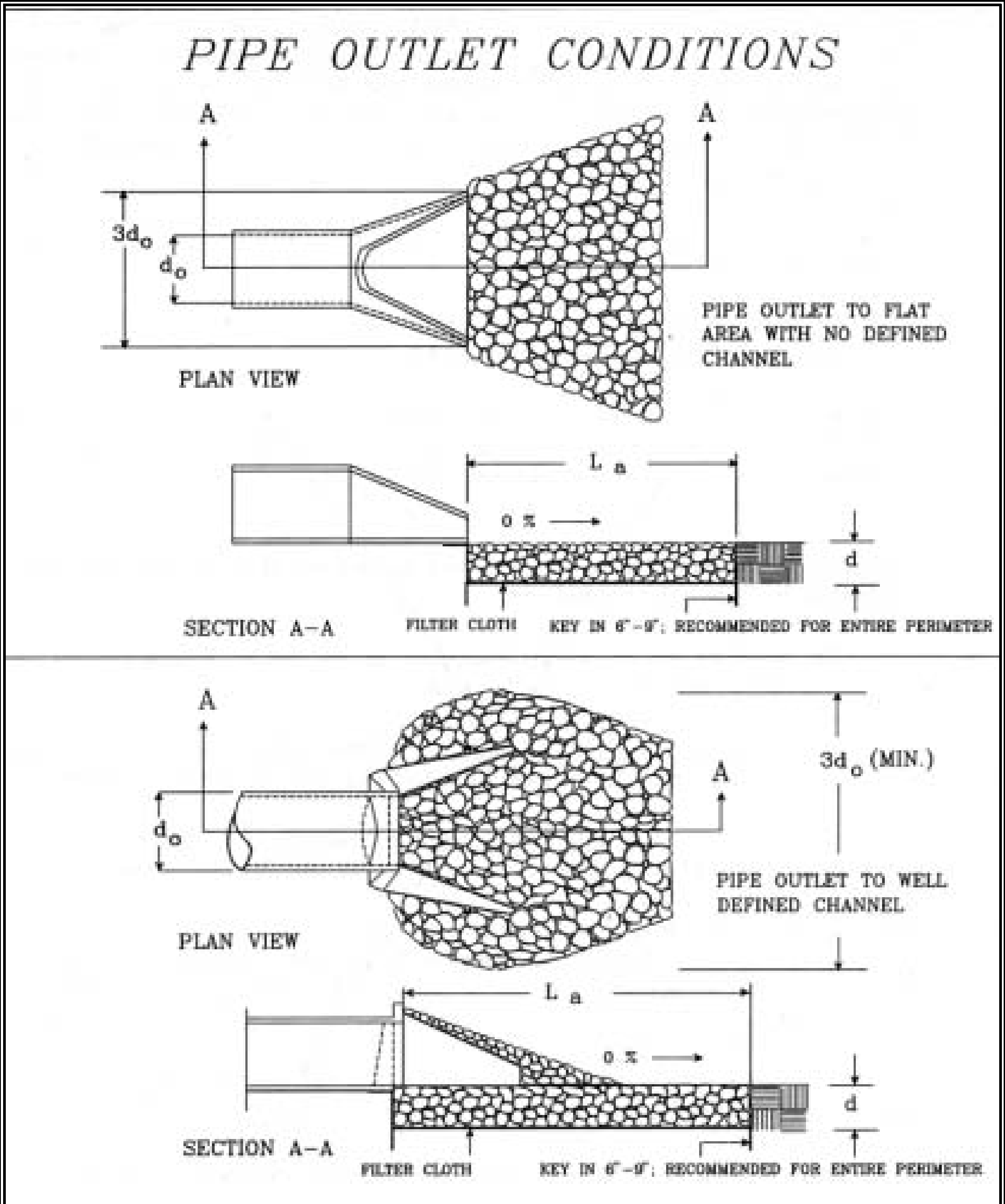
Additional design considerations and specifications can be found in **Minimum Standard 3.02, Principal Spillway** and Std. and Spec. 3.18 and 3.19 of the Virginia Erosion and Sediment Control Handbook, 1992 edition.

TABLE 5 - 11
Graded Riprap Design Values

Riprap Class	D_{15} Weight *(lbs.)*	Mean D_{15} Spherical Diameter *(ft.)*	Mean D_{50} Spherical Diameter *(ft.)*
Class AI	25	0.7	0.9
Class I	50	0.8	1.1
Class II	150	1.3	1.6
Class III	500	1.9	2.2
Type I	1,500	2.6	2.8
Type I	6,000	4.0	4.5

Source: VDOT Drainage Manual

FIGURE 5 - 19
Outlet Protection Detail



NOTES:
1. APRON LINING MAY BE RIPRAP, GROUTED RIPRAP, GABION BASKET, OR CONCRETE.
2. La IS THE LENGTH OF THE RIPRAP APRON AS CALCULATED USING FIG 5-20 AND FIG 5-21.
3. d = 1.5 TIMES THE MAXIMUM STONE DIAMETER, BUT NOT LESS THAN 6 INCHES.

FIGURE 5 - 20
Minimum Tailwater Condition

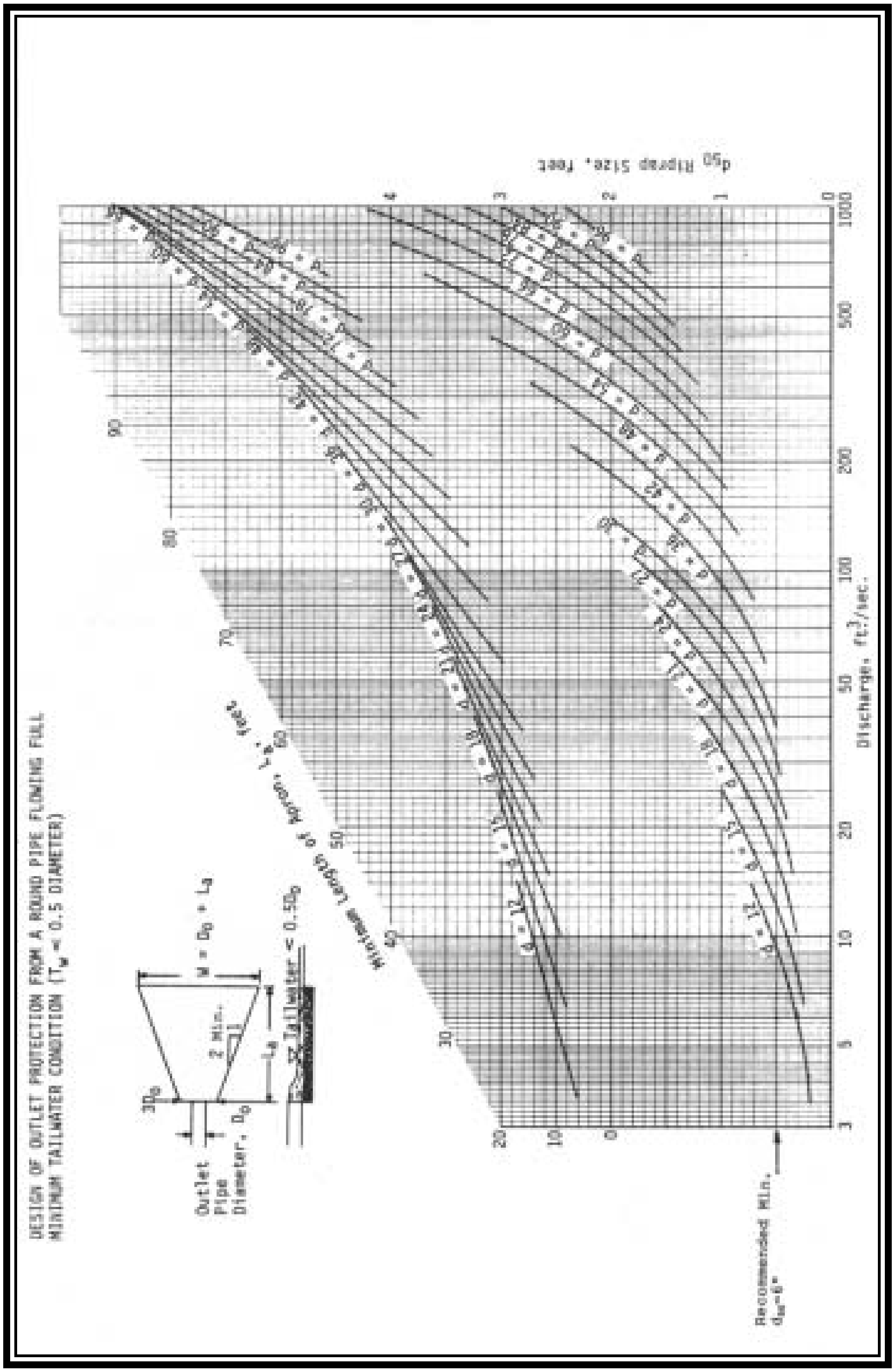
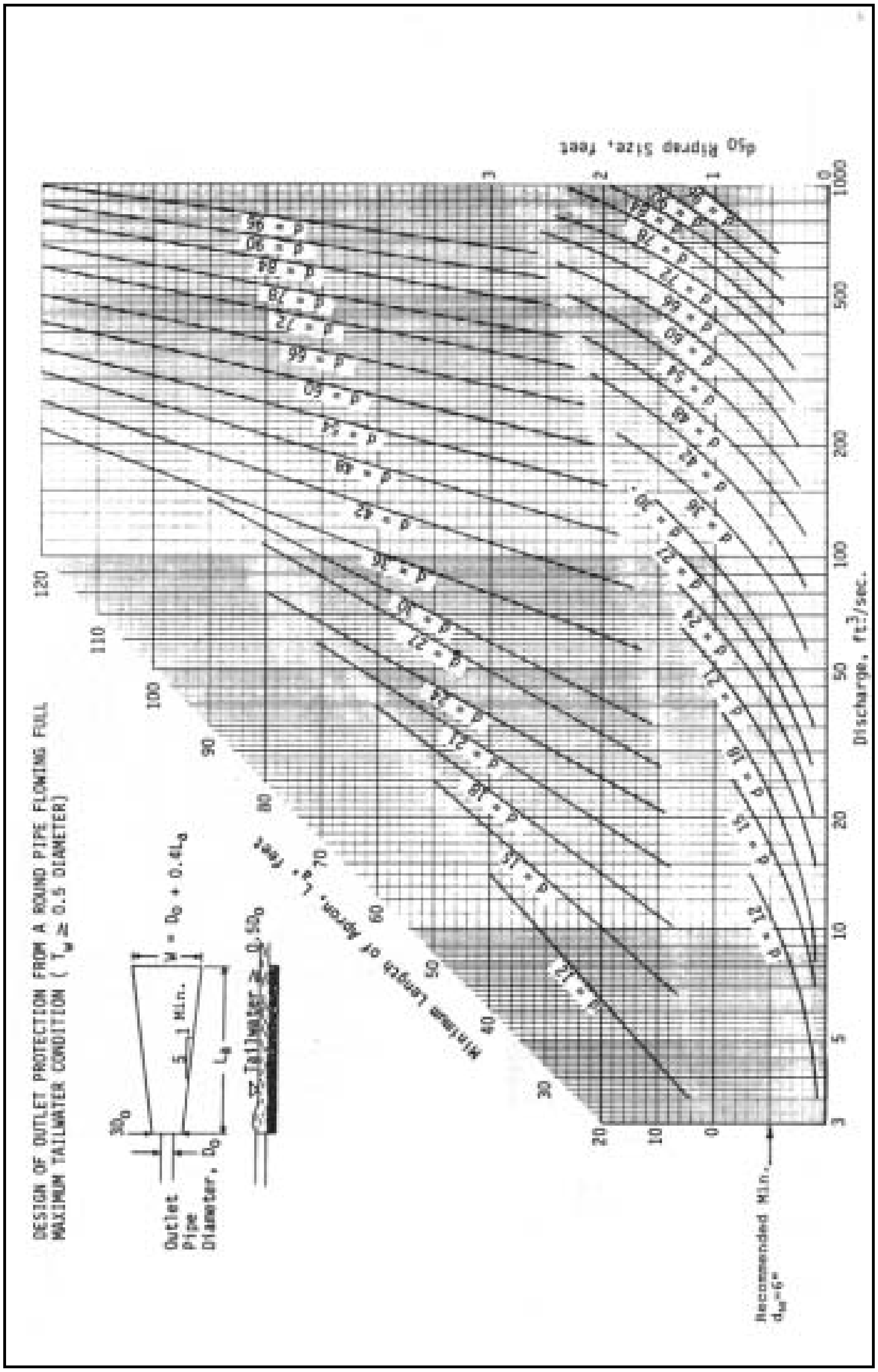


FIGURE 5 - 21
Maximum Tailwater Condition



STEP 14 Perform Buoyancy Calculation

The design of a multi-stage riser structure must include a buoyancy analysis for the riser and footing. When the ground is saturated and ponded runoff is at an elevation higher than the footing of the riser structure, the riser structure acts like a vessel. During this time, the riser is subject to uplifting, buoyant forces that are relative in strength to the volume of water displaced. **Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water**. Flotation forces on the riser can lead to failure of the connection between the riser structure and barrel, and any other rigid connections. Eventually, this can also lead to the failure of the embankment.

A buoyancy calculation is the summation of all forces acting on the riser. The upward force is the weight of the water, or 62.4 lb/ft^3. The downward force includes the weight of the riser structure, any components, such as trash racks, and the weight of the soil above the footing. Note that conventional reinforced concrete weighs about 150 lb/ft^3 and the unit weight of soil is approximately 120 lb/ft^3. The weight of components such as trash racks, anti-vortex devices, hoods, etc. is very specific to each structure and, depending upon the design, may or may not be significant in comparison to the other forces. If an extended base footing is used below the ground surface to support the control structure, then the weight of the soil above the footing may also be a significant force.

The outlet pipe is excluded from the buoyancy analysis for the control structure. However, the barrel should be analyzed separately to insure that it is not subject to flotation. The method used to attach the control structure to the outlet pipe is considered to have no bearing on the potential for these components to float.

The following procedure compares the upward force (buoyant force) to the downward force (structure weight). To maintain adequate stability, **the downward force should be a minimum of 1.25 times the upward force**.

1. Determine the buoyant force.

The buoyant force is the total volume of the riser structure and base, using outside dimensions (i.e., *the total volume displacement of the riser structure*) multiplied by the unit weight of water (62.4 lb/ft^3).

2. Determine the downward or resisting force.

The downward force is the total volume of the riser walls below the crest, including any top slab, footing, etc., less the openings for any pipe connections, multiplied by the unit weight of reinforced concrete (150 lb/ft^3). Additional downward forces from any components may also be added, including the weight of the soil above the extended footing.

3. Decide if the downward force is greater than the buoyant force by a factor of 1.25 or more.

If the downward force is not greater than the buoyant force by a factor of 1.25 or more, then additional weight must be added to the structure. This can be done by sinking the riser footing deeper into the ground and adding concrete to the base. Note that this will also increase the buoyant force, but since the unit weight of concrete is more than twice that of water, the net result will be an increase in the downward force. The downward and buoyant forces should be adjusted accordingly, and step 3 repeated.

STEP 15 Provide Seepage Control

Seepage control should be provided for the pipe through the embankment. The two most common devices for controlling seepage are 1) *filter and drainage diaphragms* and 2) *anti-seep collars*. The use of these devices is discussed in detail in **Minimum Standard 3.02, Principal Spillway**. Note that filter and drainage diaphragms are preferred over anti-seep collars for controlling seepage along pipe conduits.

a. Filter & Drainage Diaphragms

The design of filter and drainage diaphragms depends on the foundation and embankment soils and is outside the scope of this manual. When filter and drainage diaphragms are warranted, their design and construction should be supervised by a registered professional engineer. Design criteria and construction procedures for filter and drainage diaphragms can be found in the following references:

- USDA SCS TR-60
- USDA SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters*
- USDA SCS Soil Mechanics Note No. 3: Soil Mechanics Consideration for Embankment Drains*
- U.S. Department of the Interior ACER Technical Memorandum No. 9: Guidelines for Controlling Seepage Along Conduits Through Embankments

* *These publications include design procedures and examples and are provided in* ***Appendix 5B****.*

b. Anti-Seep Collars

The Bureau of Reclamation, the U.S. Army Corps of Engineers and the Soil Conservation Service no longer recommend the use of anti-seep collars. In 1987, the Bureau of Reclamation issued Technical Memorandum No. 9 that states:

> *"When a conduit is selected for a waterway through an earth or rockfill embankment, cutoff [anti-seep]collars will not be selected as the seepage control measure."*

Alternative measures to anti-seep collars include *graded filters* (or *filter diaphragms*) and *drainage blankets*. These devices are not only less complicated and more cost-effective to construct than cutoff collars, but also allow for easier placement of the embankment fill. Despite this information, anti-seep collars may be appropriate for certain situations. A design procedure is provided below. Criteria for the use and placement of anti-seep collars are presented in **Minimum Standard 3.02, Principal Spillway**.

1. Determine the length of the barrel within the saturated zone using the following equation:

$$L_s = Y\,(Z + 4)\quad 1 + \frac{S}{0.25 - S}$$

Equation 5 - 11
Barrel Length in Saturated Zone

Where:
L_s = *length of the barrel in the saturated zone, ft.*
Y = *the depth of water at the principal spillway crest (10-year frequency storm water surface elevation), ft.*
Z = *slope of the upstream face of the embankment, in Z ft. horizontal to 1 ft. vertical (Z ft. H:1V).*
S = *slope of the barrel, in feet per foot.*

The length of pipe within the saturated zone can also be determined graphically on a *scale profile* of the embankment and barrel. The saturated zone of the embankment can be approximated as follows: starting at a point where the 10-year storm water surface elevation intersects the embankment slope, extend a line at a *4H:1V* slope downward until it intersects the barrel. The area under this line represents the ***theoretical zone of saturation*** (refer to **Figure 5-22**).

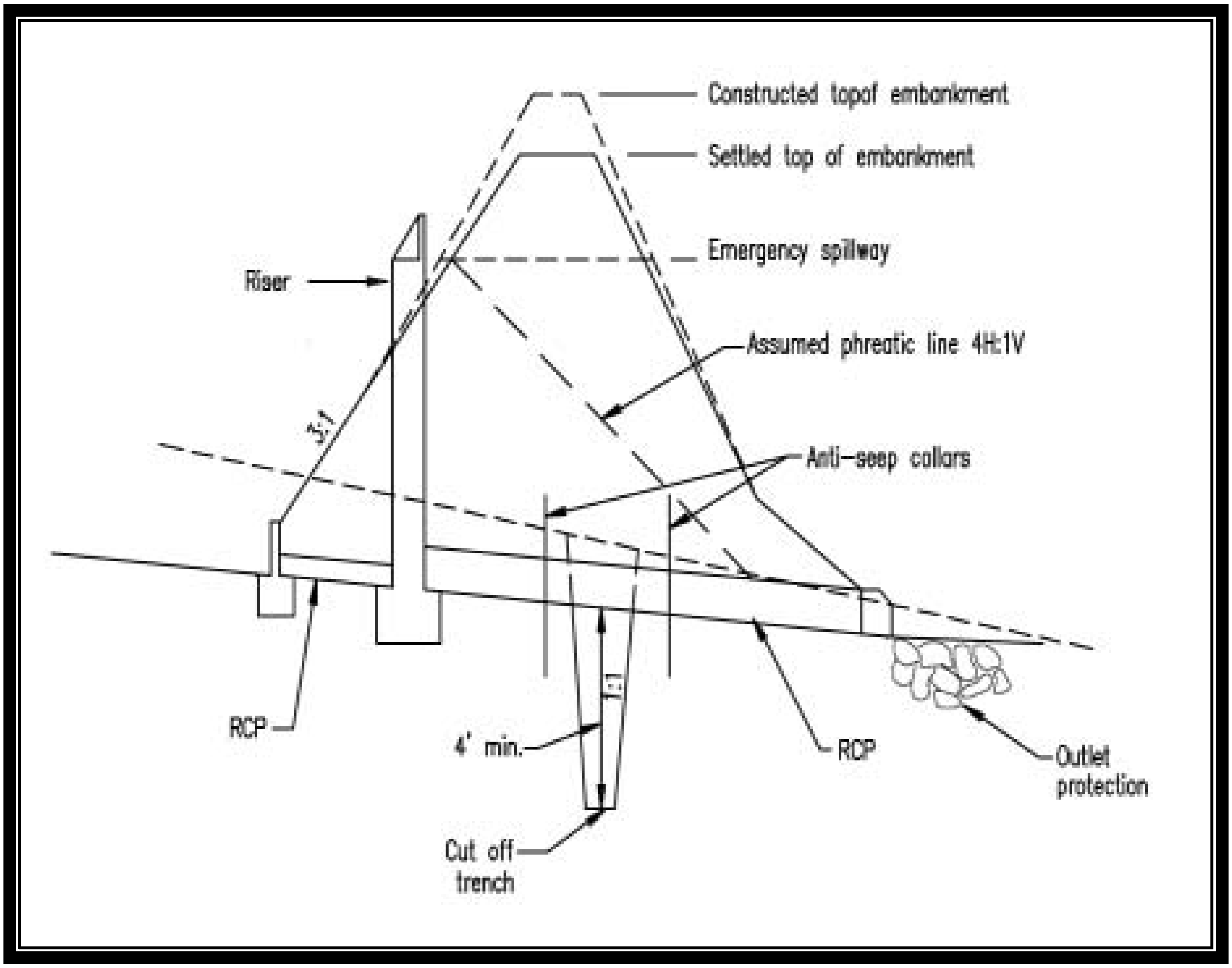
2. Determine the length required by multiplying 15% times the seepage length: $0.15\ L_s$. The increase in seepage length represents the total collar projection. This can be provided for by one or multiple collars.

3. Choose a collar size that is at least 4 feet larger than the barrel diameter (2 feet above and 2 feet below the barrel). For example, a 7-feet square collar would be selected for a 36-inch diameter barrel.

4. Determine the collar projection by subtracting the pipe diameter from the collar size.

5. Determine the number of collars required. The number of collars is found by dividing the seepage length increase, found in Step 2, by the collar projection from Step 4. To reduce the number of collars required, the collar size can be increased. Alternatively, the collar size can be decreased by providing more collars.

FIGURE 5 - 22
Phreatic Line Graphical Determination



SUMMARY
MULTI-STAGE RISER DESIGN PROCEDURE

STEP 1:	Determine Water Quality Volume Requirements a. Extended-Detention b. Retention
STEP 2:	Compute Allowable Release Rates
STEP 3:	Estimate the Required Storage Volume
STEP 4:	Grade the Basin; Create Stage-Storage Curve
STEP 5a:	Design Water Quality Orifice (Extended-Detention)
STEP 5b:	Set Permanent Pool Volume (Retention)
STEP 6:	Size 2-Year Control Orifice
STEP 7:	Check Performance of 2-Year Opening
STEP 8:	Size 10-Year Control Opening
STEP 9:	Check Performance of 10-Year Opening
STEP 10:	Perform Hydraulic Analysis a. Riser Flow Control b. Barrel Flow Control 1. Barrel Inlet Flow Control 2. Barrel Outlet Flow Control
STEP 11:	Size 100-Year Release Opening or Emergency Spillway
STEP 12:	Calculate Total Discharge and Check Performance of 100-Year Control Opening
STEP 13:	Design Outlet Protection
STEP 14:	Perform Buoyancy Calculation
STEP 15:	Provide Seepage Control

5-8 EMERGENCY SPILLWAY DESIGN

A vegetated emergency spillway is designed to convey a predetermined design flood volume without excessive velocities and without overtopping the embankment.

Two design methods are presented here. The first (Procedure 1) is a conservative design procedure which is also found in The Virginia Erosion & Sediment Control Handbook, 1992 edition, Std. & Spec. 3.14. This procedure is typically acceptable for stormwater management basins. The second method (Procedure 2) utilizes the roughness, or retardance, and durability of the vegetation and soils within the vegetated spillway. This second design is appropriate for larger or regional stormwater facilities where construction inspection and permanent maintenance are more readily enforced. These larger facilities typically control relatively large watersheds and are located such that the stability of the emergency spillway is essential to safeguard downstream features.

The following design procedures establish a stage-discharge relationship (H_p versus Q) for a vegetated emergency spillway serving a stormwater management basin (refer to **Figure 5-23**).

The information required for these designs includes the determination of the hydrology for the watershed draining to the basin. Any of the methods, as outlined in **Chapter 4**, may be used. The design should include calculations for the ***allowable release rate*** from the basin if the spillway is to be used to control a design frequency storm. Otherwise, the ***design peak flow rate*** should be calculated based on the spillway design flood, or downstream conditions.

(In general, a vegetated emergency spillway should not be used as an outlet for any storm less than the 100-year frequency storm, unless it is armored with a non-erodible material. The designer must consider the depth of the riprap blanket when riprap is used to armor the spillway. As noted previously, Class I riprap would require a blanket thickness or stone depth of 30" which may add considerable height to the embankment.)

> ***The design maximum water surface elevations for the emergency spillway should be at least 1 foot lower than the settled top of the embankment. Refer to Minimum Standard 3.03, Vegetated Emergency Spillways.***

Procedure 1:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin using the developed condition hydrology **or** determine the ***allowable design peak release rate, Q,*** from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the maximum flow through the

riser and barrel system at this elevation (refer to the stage-storage-discharge table). Subtract this flow volume from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Determine the crest elevation of the emergency spillway. This is usually a small increment (0.1 feet) above the design high water elevation of the next smaller storm, typically the 10-year frequency storm.

4. Enter **Table 5-12** with the maximum *Hp* value (maximum design water surface elevation from Step 2, less the crest elevation of the emergency spillway), and read across for the desired maximum spillway design discharge (from Step 2 above). Read the design bottom width of the emergency spillway (in feet) at the top of the table, and verify the minimum exit slope (s) and length (x), **or**;

   If a maximum bottom width (b) is known due to grading or topographic constraints, enter **Table 5-12** at the top with the desired bottom width and read down to find the desired discharge, Q, and then read across to the left to determine the required flow depth, *Hp*.

5. Add the appropriate *Hp* and discharge Q values to the stage-storage-discharge table.

Example Procedure 1:

Given: Q = *250 cfs* (determined from post-developed condition hydrology)
s_o = *4%* (slope of exit channel)
L = *50 ft.*(length of level section)

Find: Width of spillway, b, velocity, v, and depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 5 of design **Procedure 1** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: <u>given</u> Q = *250 cfs.*

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *163 cfs*. The desired maximum spillway design discharge is *250 cfs - 163 cfs = 87 cfs*, at a H_p value of *1.3 ft.*

3. Emergency spillway excavated into undisturbed material. The slope of the exit channel and length and elevation of level section: <u>given</u>, s_o = *4%*, L = *50 ft.*, elevation = 100.0' (given).

4. Enter **Table 5-12** with the desired H_p value of *1.3 ft.* And read across to *86 cfs*, and read up to a bottom width of *24 ft*. at the top of the table. The minimum exit channel slope is *2.7%* which

is less than the *4%* provided, and the length of exit channel is required to be *63 ft*. The velocity within the exit channel is *4.7 ft/s* at an exit channel slope of *2.7%*. Since the provided exit channel slope is *4.0%*, erosive velocities may warrant special treatment of the exit channel.

5. Add the elevation corresponding to *1.3 ft*. above the crest of the emergency spillway to the Stage-Storage-Discharge Worksheet.

Procedure 2:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin, using the developed condition hydrology, **<u>or</u>** determine the ***allowable design peak release rate*** , Q, from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the associated flow through the riser and barrel system for this elevation. Subtract this flow value from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Position the emergency spillway on the basin grading plan at an embankment abutment.

4. Determine the slope, s_o , of the proposed exit channel, and the length, L, and elevation of the proposed level section from the basin grading plan.

5. Classify the natural soils around the spillway as ***erosion resistant*** or ***easily erodible*** soils.

6. Determine the type and height of vegetative cover to be used to stabilize the spillway.

7. Determine the permissible velocity, v, from **Table 3-03.1**, based on the vegetative cover, soil classification, and the slope of the exit channel, s_o .

8. Determine the retardance classification of the spillway based on the type and height of vegetative cover from **Table 3-03.2**.

9. Determine the unit discharge of the spillway, q, in *cfs/ft*, from **Table 5-13(a-d)** for the selected retardance, the maximum permissible velocity, v, and the slope of the exit channel, s_o .

10. Determine the required bottom width of the spillway, in *ft.*, by dividing the allowable or design discharge, Q , by the spillway unit discharge, q:

$$\frac{Q(cfs)}{q(cfs/ft)} \quad ft.$$

11. Determine the depth of flow, H_p , upstream of the control section based on the length of the

level section, L, from **Table 5-13(a-d)**.

12. Enter the stage-discharge information into the stage-storage-discharge table.

The following examples use **Tables 3-03.1, 3-03.2** and **5-13** to find the capacity of a vegetated emergency spillway.

Example Procedure 2:

Given: $Q = 250\ cfs$ (determined from post-developed condition hydrology)
$s_o = 4\%$ (slope of exit channel)
$L = 50\ ft.$(length of level section)
Erosion resistant soils
Sod forming grass-legume mixture cover, 6 to 10-inch height

Find: Permissible velocity, v, width of spillway, b, depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 12 of design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: given $Q = 250\ cfs$.

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be $163\ cfs$. The desired maximum spillway design discharge is $250\ cfs - 163\ cfs = 87\ cfs$.

3. Emergency spillway excavated into undisturbed material.

4. Slope of exit channel, and length and elevation of level section: given, $s_o = 4\%$, $L = 50\ ft.$, elevation = 100.0 feet (given).

5. Soil classification: given, erosion resistant soils.

6. Vegetative cover: given, sod-forming grass-legume mixture.

7. Permissible velocity $v = 5\ ft/s$ from **Table 3-03.1** for sod-forming grass-legume mixtures, erosion resistant soils, and exit channel slope $s_o = 4\%$.

8. Retardance classification, C, from **Table 3.03.2** for sod-forming grass-legume mixtures, *expected height = 6 to 10 inches*.

9. The unit discharge of the spillway $q = 3\ cfs/ft$ from **Table 5-13c** for Retardance C, maximum permissible velocity $v = 5\ ft/s$, and exit channel slope $s_o = 4\%$.

10. The required bottom width $b = Q \quad q = 87\ cfs/\ 3\ \ cfs/ft = 29\ ft$.

11. The depth of flow, $H_{p,}$ from **Table 5-13c** for Retardance C ; enter at $q = 3\ cfs/ft$, find $H_p = 1.4\ ft.$ for level section $L = 50\ ft.$

12. The stage-discharge relationship: at stage elevation 1.4 feet above the spillway crest (*101.4'*), the discharge is 87 *cfs*.

Example Procedure 2:

Given: $Q\ = 175\ cfs$ (determined from post-developed hydrology)
$s_o\ =\ 8\ \%$ (slope of exit channel)
$L\ =\ 25\ ft.$ (length of level section)
Easily erodible soil
Bahiagrass, good stand, 11 to 24 inches expected

Find: Permissible velocity, v, width of spillway, b, depth of water above the spillway crest, H_p.
Analyze the spillway for stability during the vegetation establishment period, and capacity once adequate vegetation is achieved.

Solution: Complete Steps 1 through 12 of the design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. $Q = 175\ cfs.$

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *75 cfs*. The desired maximum spillway design discharge is $175\ cfs - 75\ cfs = 100\ cfs$.

3. Emergency spillway in undisturbed ground.

4. $s_o = 8\ \%;\quad L = 25\ ft.$, elevation = *418.0 feet* (given)

5. Easily erodible soils.

6. Bahiagrass, good stand, 11 to 24 inches expected.

7. Permissable velocity, $v = 5\ ft/s$, from **Table 3-03.1**.

8. a) Retardance used for **stability** during the establishment period - good stand of vegetation 2 to 6 inches; Retardance D**.**

   b) Retardance used for **capacity** - good stand of vegetation 11 to 24 inches; Retardance B**.**

9. Unit discharge $q = 2\ cfs/ft$ for stability. From **Table 5-13d** for Retardance D, permissable velocity, $v = 5\ ft/s.$, and $s_o = 8\%$

10. Bottom width $b = Q/\ q = 100\ cfs/\ 2\ cfs/\ ft = 50\ ft.$ (stability)

11. The depth of flow, $H_{p,}$ for capacity. From **Table 5-13b** for Retardance B, enter at $q = 2\ cfs/ft$, find $H_p = 1.4\ ft.$ for $L = 25\ ft.$

12. The stage-discharge relationship: at stage (elevation) 1.4 *ft.* above the spillway crest (*419.4'*), the discharge, *Q*, is 100 *cfs*.

TABLE 5-12
Design Data for Earth Spillways

STAGE (H_p) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	55	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	65	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	35	40	45	51	58	64	69	75	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	83	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.5	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	156	175	193	208	226	243	258	275	292	306	323	341	354
	V	6.0	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

Source: USDA - SCS

TABLE 5 - 13a

H_p and Slope Range for Discharge, Velocity and Crest Length - Retardance A

Max. Velocity, *v* (*ft/s*)	**Unit Discharge, *q* (*cfs/ft*)**	**Depth of Water Above Spillway Crest, H_p (*ft.*) — Length of Level Section, *L* (*ft.*)**				**Slope Range, s_o %**	
		25	**50**	**100**	**200**	**Min.**	**Max.**
3	3	2.3	2.5	2.7	3.1	1	11
4	4	2.3	2.5	2.8	3.1	1	12
4	5	2.5	2.6	2.9	3.2	1	7
5	6	2.6	2.7	3.0	3.3	1	9
6	7	2.7	2.8	3.1	3.5	1	12
7	10	3.0	3.2	3.4	3.8	1	9
8	12.5	3.3	3.5	3.7	4.1	1	10

Source: SCS Engineering Field Manual

TABLE 5 - 13b

H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance B

Max. Velocity, *v* (*ft/s*)	Unit Discharge, *q* (*cfs/ft*)	Depth of Water Above Spillway Crest, H_p (*ft.*) — Length of Level Section, *L (ft.)*				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	1	1.2	1.4	1.5	1.8	1	12
2	1.25	1.3	1.4	1.6	1.9	1	7
3	1.5	1.3	1.5	1.7	1.9	1	12
3	2	1.4	1.5	1.7	1.9	1	8
4	3	1.6	1.7	1.9	2.2	1	9
5	4	1.8	1.9	2.1	2.4	1	8
6	5	1.9	2.1	2.3	2.5	1	10
7	6	2.1	2.2	2.4	2.7	1	11
8	7	2.2	2.4	2.6	2.9	1	12

Source: SCS Engineering Field Manual

TABLE 5 - 13c

H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance C

Max. Velocity, *v* (*ft/s*)	Unit Discharge, *q* (*cfs/ft*)	Depth of WaterAbove Spillway Crest, H_p (*ft.*) — Length of Level Section, *L (ft.)*				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	0.5	0.7	0.8	0.9	1.1	1	6
2	1	0.9	1.0	1.2	1.3	1	3
3	1.25	0.9	1.0	1.2	1.3	1	6
4	1.5	1.0	1.1	1.2	1.4	1	12
4	2	1.1	1.2	1.4	1.6	1	7
5	3	1.3	1.4	1.6	1.8	1	6
6	4	1.5	1.6	1.8	2.0	1	12
8	5	1.7	1.8	2.0	2.2	1	12
9	6	1.8	2.0	2.1	2.4	1	12
9	7	2.0	2.1	2.3	2.5	1	10
10	7.5	2.1	2.2	2.4	2.6	1	12

Source: SCS Engineering Field Manual

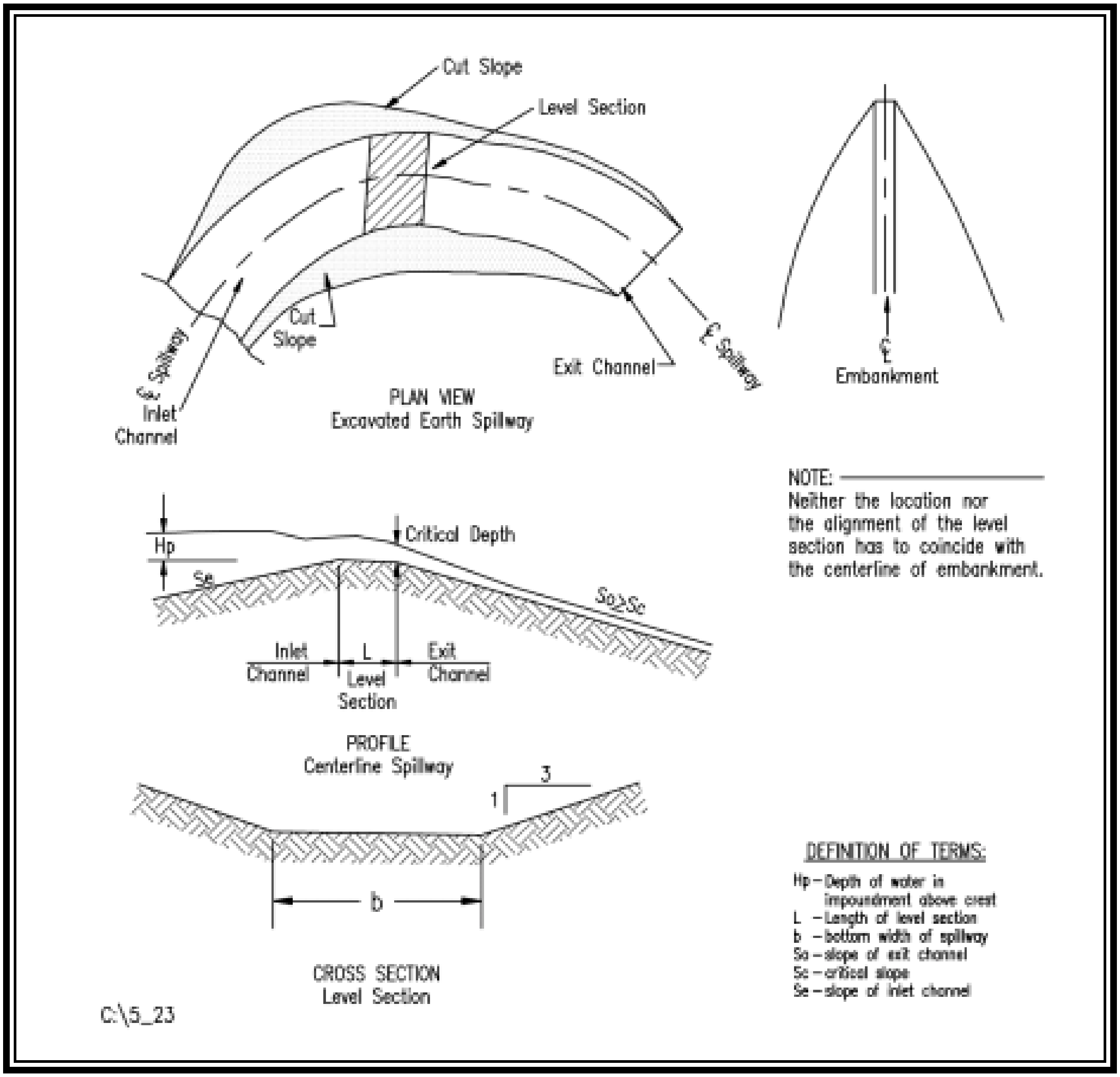
TABLE 5 - 13d

H_p and Slope Range for Discharge, Velocity, and Crest Length, Retardance D

Max. Velocity, *v* (*ft/s*)	**Unit Discharge, *q* (*cfs/ft*)**	**Depth of Water Above Spillway Crest, H_p (*ft.*) — Length of Level Section, *L (ft.)***				**Slope Range, s_o %**	
		25	**50**	**100**	**200**	**Min.**	**Max.**
2	0.5	0.6	0.7	0.8	0.9	1	6
3	1	.8	.9	1.0	1.1	1	6
3	1.25	.8	.9	1.0	1.2	1	4
4	1.5	.8	.9	1.0	1.2	1	10
4	2	1.0	1.1	1.3	1.4	1	4
5	1.5	.9	1.0	1.2	1.3	1	12
5	2	1.0	1.2	1.3	1.4	1	9
5	3	1.2	1.3	1.5	1.7	1	4
6	2.5	1.1	1.2	1.4	1.5	1	11
6	3	1.2	1.3	1.5	1.7	1	7
7	3	1.2	1.3	1.5	1.7	1	12
7	4	1.4	1.5	1.7	1.9	1	7
8	4	1.4	1.5	1.7	1.9	1	12
8	5	1.6	1.7	1.9	2.0	1	8
10	6	1.8	1.9	2.0	2.2	1	12

Source: SCS Engineering Field Manual

FIGURE 5 - 23
Vegetated Emergency Spillways: Typical Plan and Section



5-9 HYDROGRAPH ROUTING

This section presents the methodology for routing a runoff hydrograph through an existing or proposed stormwater basin. The "level pool" or storage indication routing technique is one of the simplest and most commonly used methods, and is based on the continuity equation:

$$I \quad - \quad O \quad = \quad ds / dt$$

Inflow - Outflow = Change in Storage over time

The goal of the routing process is to create an outflow hydrograph that is the result of the combined effects of the outlet device and the available storage. This will allow the designer to evaluate the performance of the outlet device or the basin storage volume, or both. When multiple iterations are required to create the most efficient basin shape, the routing procedure can be time consuming and cumbersome, especially when done by hand using the methods presented in this section. It should be noted that several computer programs are available to help complete the routing procedure.

A step-by-step procedure for routing a runoff hydrograph through a stormwater basin is given below. Note that the first four steps are part of the multi-stage riser design of the previous section. Due to the complexity of this procedure, **Example 1** from **Chapter 6** will be used. Note that the water quality volume is **not** considered and only one design storm will be routed, the 2-year storm. Other design frequency storms can be easily analyzed with the same procedure. Blank worksheets for this procedure are provided in **Appendix 5D**.

Procedure:

1. Generate a post-developed condition inflow hydrograph. The runoff hydrograph for the 2-year frequency storm, post-developed condition from **Example 1**, as calculated by the SCS TR-20 computer program and shown in **Figure 5-1**, will be used for the inflow hydrograph. (Refer to **Chapter 6** for details on the hydrology from **Example 1**. Refer to **Chapter 4** for information on the hydrologic methods used.)

2. Develop the stage-storage relationship for the proposed basin. The hydrologic calculations and the hydrograph analysis for **Example 1**, in **Section 5-3** and **Section 5-4.1**, revealed that the storage volume required to reduce the 2-year, post-developed peak discharge back to the pre-developed rate was 35,820 ft^3. Therefore, a preliminary grading plan should have a stormwater basin with this required storage volume, as a minimum, to control the 2-year frequency storm. The stage-storage relationship of the proposed stormwater facility can be generated by following the procedures outlined in **Section 5-5**. **Figure 5-10** shows the completed Storage Volume Calculations Worksheet, and **Figure 5-11** shows the stage vs. storage curve.

3. Size the outlet device for the design frequency storm and generate the stage-discharge relationship. An outlet device or structure must be selected to define the stage-discharge relationship. This procedure is covered in **Section 5-7, STEP 6** of the multi-stage riser design. Using the procedure within **STEP 6** from **Section 5-7** and **Example 1**, the procedure is as follows (from **STEP 6, Section 5-7**):

    1. Approximate the 2-year maximum head, $h_{2_{max}}$.

    Enter the stage-storage curve, **Figure 5-11**, with the 2-year required storage: 35,820 ft^3 and read the corresponding elevation: 88.5 *ft*. Then, $h_{2_{max}}$ *= 88.5 ft. - 81.0 ft. (bottom of basin) = 7.5 ft*. Note that this is an approximation because it ignores the centerline of the orifice as the point from which the head is measured. The head values can be adjusted when the orifice size is selected.

    2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$.

    From the pre-developed hydrologic analysis, the 2-year allowable discharge from the basin was found to be 8.0 *cfs*. (This assumes that watershed conditions or local ordinance limit the developed rate of runoff to be the pre-developed rate.)

    3. Calculate the size of the 2-year controlled release orifice.

    Solve for the area, *a*, in ft^2 by inserting the allowable discharge $Q = 8.0\ cfs$ and $h_{2_{max}} = 7.5\ ft.$ into the **Rearranged Orifice Equation**, **Equation 5-7**. This results in an orifice diameter of 10 inches.

$$a \qquad \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

Where:
a = *required orifice area,* ft^2
Q = *maximum allowable discharge = 8.0 cfs*
C = *orifice coefficient = 0.6*
g = *gravitational acceleration = 32.2 ft/sec*
h = *maximum 2-year hydraulic head,* $h_{2_{max}} = 7.5\ ft.$

$$a \qquad \frac{8.0}{0.6\sqrt{(2)(32)(7.5)}}$$

$$a \;\; = 0.61\ ft^2$$

For orifice diameter:

$$a \qquad 0.61\ \ ft^2 \qquad ´ \left(\frac{d}{2}\right)^2$$

$$d = 0.88\ ft. = 10.6\ inches$$

***Use a 10-inch diameter orifice*.**

4. Develop the stage-storage-discharge relationship for the 2-year storm.

   Substituting the 10-inch orifice size into the **Orifice Equation**, **Equation 5-6**, and solving for the discharge, *Q*, at various stages provides the information needed to plot the stage vs. discharge curve and complete the Stage-Storage-Discharge Worksheet.

$$Q \qquad C_o a\sqrt{2gh}$$

Equation 5-6
Orifice Equation

Where: $a = a_{10''} = 0.545\ ft^2$

$$Q \qquad (0.6)(0.545)\sqrt{(2)(32.2)(h)}$$

$$Q_2 = 2.62\ (h)^{0.5}$$

Where: h = *water surface elev. - (81.0 + 0.83/2)*
h = *water surface elev. - 81.4*

Note that the *h* is measured to the centerline of the 10-inch orifice.

Figure 5-24 shows the result of the calculations: the stage vs. discharge curve and table.

Continuing with the Hydrograph Routing Procedure:

5. Develop the relationship *2S/ϕt vs. O* and plot *2S/ϕt vs. O.*

The plot of the curve *2S/ϕt vs. 0* is derived from the continuity equation. The continuity equation is rewritten as:

$$\frac{I_n \quad I_{n\ 1}}{2} \qquad \frac{O_n \quad O_{n\ 1}}{2} \qquad \frac{S_{n\ 1} \quad S_n}{\varphi t}$$

Equation 5-12
Continuity Equation

where: I_n & I_{n+1} = *inflow at time n=1 and time n=2*
O_n & O_{n+1} = *outflow at time n=1 and time n=2*
S_n & S_{n+1} = *storage at time n=1 and time n=2*
φt = *time interval (n=2 - n=1)*

This equation describes the *change in storage over time* as the difference between the average inflow and outflow at that given time. Multiplying both sides of the equation by 2 and rearranging allows the equation to be re-written as:

$$I_n \qquad I_{n\ 1} \qquad \left(\frac{2S_n}{\varphi t} \quad O_n \right) \qquad \frac{2S_{n\ 1}}{\varphi t} \qquad O_{n\ 1}$$

Equation 5-13
Rearranged Continuity Equation

The terms on the left-hand side of the equation are known from the inflow hydrograph and from the storage and outflow values of the previous time interval. The unknowns on the right hand side, O_{n+1} and S_{n+1} , can be solved interactively from the previously determined stage vs. storage curve, **Figure 5-11**, and stage vs. discharge curve, **Figure 5-24**.

First, however, the relationship between *2S/ϕt + O* and *O* must be developed. This relationship can best be developed by using the stage vs. storage and stage vs. discharge curves to fill out the worksheet shown in **Figure 5-25**, as follows:

a) Columns 1, 2, and 3 are completed using the stage vs. discharge curve.

b) Columns 4 and 5 are completed using the stage vs. storage curve.

c) Column 6 is completed by determining the time step increment used in the inflow hydrograph. (For Example 1, *qt = 1 hr. = 3,600 sec.*) *qt* is in seconds to create units of cubic feet per second (*cfs*) for the *2S/qt* calculation.

d) Column 7 is completed by adding Columns 3 and 6. The completed table is presented in **Figure 5-26**, and **Example 1** in **Chapter 6**, along with the plotted values from Column 3, *O* or outflow, and Column 7, *2S/qt + O*.

6. Route the inflow hydrograph through the basin and 10-inch diameter orifice. The routing procedure is accomplished by use of another worksheet, **Figure 5-27**, Hydrograph Routing Worksheet. Note that as the work is completed for each value of *n*, it becomes necessary to jump to the next row for a value. The table is completed by the following steps:

   a. Complete Column 2 and Column 3 for each time *n*. These values are taken from the inflow hydrograph. The inflow hydrograph is provided in tabular form in **Figure 5-29**. This information is either taken from the plot of the inflow hydrograph or read directly from the tabular version of the inflow hydrograph (TR-20, TR-55, etc.).

   b. Complete Column 4 for each time *n* by adding two successive inflow values from Column 3. Therefore, $Column\ 4_n = Column\ 3_n + Column\ 3_{n+1}$.

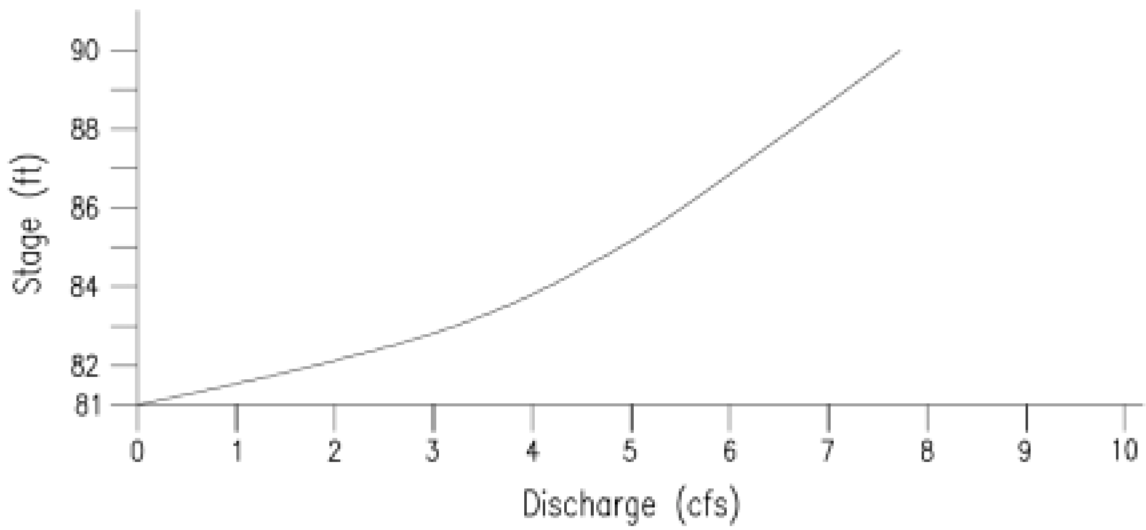
   c. Compute the values in Column 6 by adding Columns 4 and 5 from the previous time step. Note that for $n = 0$, Columns 5, 6, and 7 are given a value of zero before starting the table. Therefore, $Column\ 6_{n=2} = Column\ 4_{n=1} + Column\ 5_{n=1}$. (Note that this works down the table and not straight across.)

   d. Column 7 is read from the *2S/qt + O vs. O* curve by entering the curve with the value from Column 6 to obtain the outflow, *O*.

   e. Now backtrack to fill Column 5 by subtracting twice the value of Column 7 (from step d) from the value in Column 6. $Column\ 5_n = Column\ 6_n - 2(Column\ 7_n)$.

   f. Repeat steps c through e until the discharge (*O*, Column 7) reaches zero.

FIGURE 5 - 24
Stage vs. Discharge Curve, Example 1



Stage	(h)	(Q)
81.4	0	0
82	0.6	2.0
84	2.6	4.2
86	4.6	5.6
88	6.6	6.7
90	8.6	7.7

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FIGURE 5 - 25
Storage Indication Hydrograph Routing (2S/qt + O) vs. O Worksheet

1	2	3	4	5	6	7
elevation (ft)	***stage (ft)***	***outflow (cfs)***	***storage (cf)***	***2S (cf)***	***2S/qt (cfs)***	***2S/qt + O (cfs)***
from plan	$elev_n$ - $elev_o$	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /qt of hydrograph*	*Col 3 + Col 6*

FIGURE 5 - 26
Storage Indication Hydrograph Routing (2S/qt + O) vs. O Worksheet, Example 1, Curve & Table

1	2	3	4	5	6	7
Elev	Stage	Outflow (0 cfs)	Storage (S cf)	2S (cf)	2S/Δt (cfs)	2S/Δt + 0
81	0	0	0	0	0	0
8	0.6	2.0	900	1800	0.50	2.5
84	2.6	4.2	5940	11,880	3.3	7.5
86	4.6	5.6	14,354	28,708	7.97	13.6
88	6.6	6.7	29,582	59,164	16.4	23.1
90	8.6	7.7	55,564	111,128	30.9	38.6

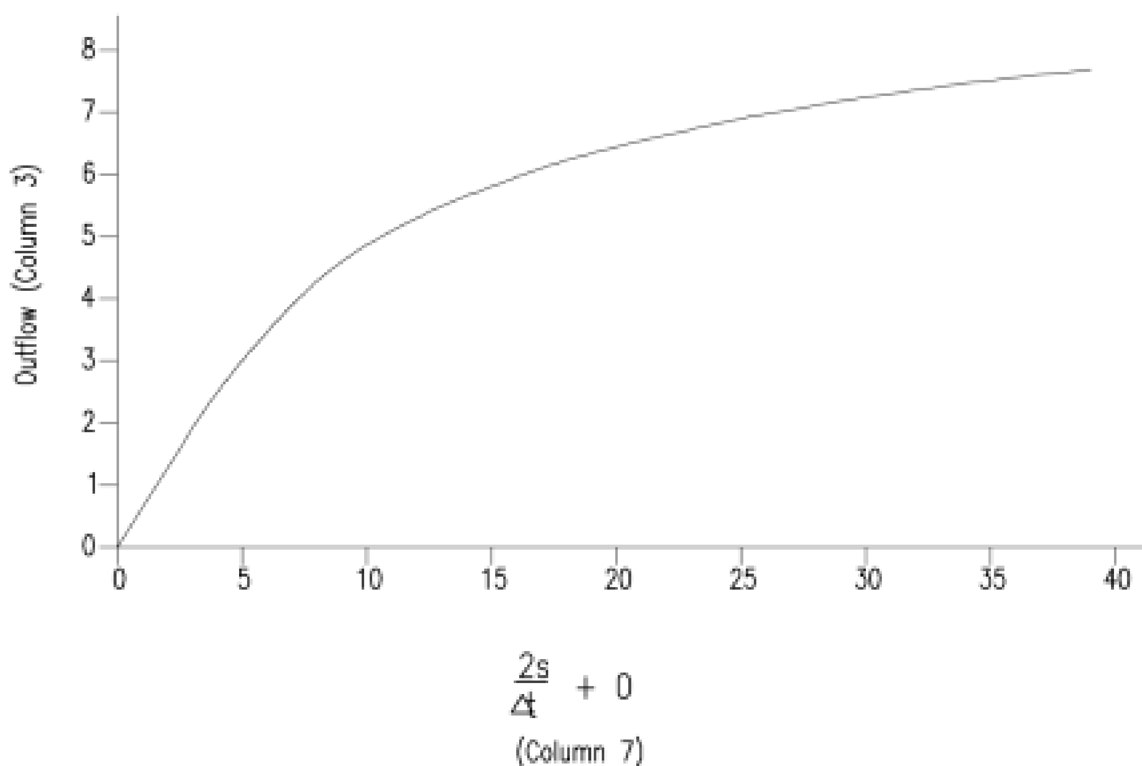

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FIGURE 5 - 27
Storage Indication Hydrograph Routing Worksheet

1	**2**	**3**	**4**	**5**	**6**	**7**
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n/\varphi t - O_n$ *(cfs)*	$2S_{n+1}/\varphi t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n *+Col* 3_{n+1}	*Col* 6_n *- 2(Col* 7_n*)*	*Col* 4_{n-1} *+ Col* 5_{n-1}	*from chart; use Col* 6_n
0				0	0	0

The above steps are repeated here for the first four time steps in **Example 1** and displayed in the completed Hydrograph Routing Worksheet, **Figure 5-28**.

1. Columns 2 and 3 are completed for each time step using the inflow hydrograph.

2. Column 4 is completed as follows:

   Column 4_n *= Column* 3_n *+ Column* 3_{n+1}
   for n = 1: Column $4_{n=1}$ *= Column* $3_{n=1}$ *+ Column* $3_{n=2}$
   Column $4_{n=1}$ *= 0 + 0.32 = 0.32*
   for n = 2: Column $4_{n=2}$ *= 0.3 + 23.9 = 24.2*
   for n = 3: Column $4_{n=3}$ *= 23.9 + 4.6 = 28.5*
   for n = 4: Column $4_{n=4}$ *= 4.6 + 2.4 = 7.0*
   etc.

n = 1

3. *Column* $6_{n=1}$ *= 0.* *n = 1* is at time 0. The first time step has a value of zero.

4. *Column* $7_{n=1}$ *= 0*. Entering the $2S/\varphi t$ *vs. O* curve with a value of zero gives *O = 0* cfs. (The discharge is always zero at time *t=0* unless a base flow exists.)

5. *Column* $5_{n=1}$ *= Column* $6_{n=1}$ *- 2 (Column* $7_{n=1}$*) Column* $5_{n=1}$ *= 0 - 0 = 0.*

n = 2

3. *Column* $6_{n=2}$ = *Column* $4_{n=1}$ + *Column* $5_{n=1}$.
   Column $6_{n=2}$ = *0.3 + 0 = 0.3.*

4. *Column* $7_{n=2}$ = *0.3*. Enter the *2S/φt + O vs. O* curve with *2S/φt + O = 0.3* (from Column 6) and read *O = 0.3.*

5. *Column* $5_{n=2}$ = *Column* $6_{n=2}$ - *2(Column* $7_{n=2}$*).*
   Column $5_{n=2}$ = *0.3 - 2(0.3) = -0.3 = 0.* (A negative outflow is unacceptable.)

n = 3

3. *Column* $6_{n=3}$ = *24.2 + 0 = 24.2.*

4. *Column* $7_{n=3}$ = *6.8.* Enter *2S/φt + O vs. O* curve with 24.2, read *O = 6.8.*

5. *Column* $5_{n=3}$ = *24.2 - 2(6.8) = 10.6.*

n = 4

3. *Column* $6_{n=4}$ = *28.5 + 10.6 = 39.1*

4. *Column* $7_{n=4}$ = *7.7.* Enter *2S/φt = O vs. O* curve with 39.1, read *O = 7.7.*

5. *Column* $5_{n=4}$ = *39.1 - 2(7.7) =23.7.*

n = 5, etc.

This process is continued until the discharge (*O*, Column 7) equals "0". The values in Column 7 can then be plotted to show the *outflow rating curve*, or *discharge hydrograph*, as shown in **Figure 5-29**. **The designer should verify that the maximum discharge from the basin is less than the allowable release**. If the maximum discharge is greater than or much less than the allowable discharge, the designer should try a different outlet size or basin shape.

FIGURE 5 - 28

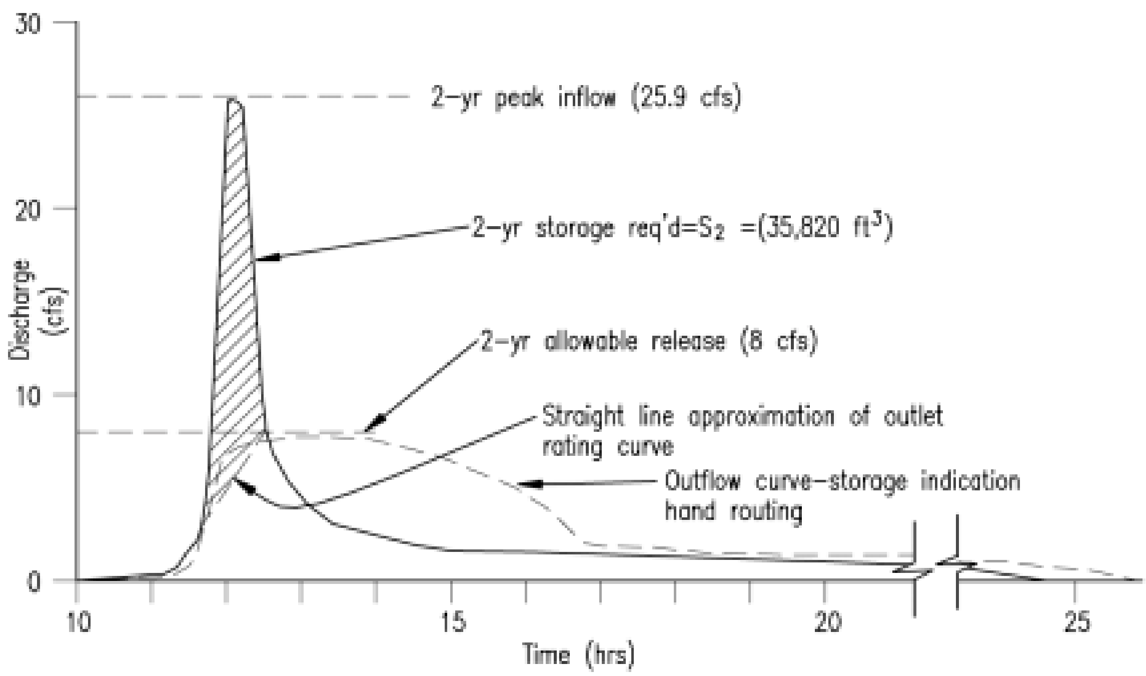
Storage Indication Hydrograph Routing Worksheet, Example 1

1	**2**	**3**	**4**	**5**	**6**	**7**
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n/\Delta t - O_n$ *(cfs)*	$2S_{n+1}/\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n *+ Col* 3_{n+1}	*Col* 6_n *- 2(Col* 7_n*)*	*Col* 4_{n-1} *+ Col* 5_{n-1}	*from chart; use Col* 6_n
1	0	0	0.32	0	0	0
2	60	0.32	24.2	0 (-0.3)	0.3	0.3
3	120	23.9	28.5	10.6	24.2	6.8
4	180	4.6	7.0	23.7	39.1	7.7
5	240	2.4	4.0	15.7	30.7	7.5
6	300	1.6	3.0	6.9	19.7	6.4
7	360	1.4	2.6	0.3	9.9	4.8
8	420	1.2	2.3	0 (-0.7)	2.9	1.8
9	480	1.1	2.1	0 (-0.7)	2.3	1.5
10	540	1.0	1.9	0 (-0.7)	2.1	1.4
11	600	0.9	1.6	0 (-0.7)	1.9	1.3
12	660	0.7	1.4	0 (-0.6)	1.6	1.1
13	720	0.7	1.4	0 (-0.6)	1.4	1.0
14	780	0.7	1.3	0 (-0.6)	1.4	1.0
15	840	0.6	0.6	0 (-0.5)	1.3	0.9
16	900	0	0	0 (-0.4)	0.6	0.5
17	960	0	0		0	0

FIGURE 5 - 29
Inflow and Discharge Hydrographs, Example 1

hr.	Qi	Qo	hr.	Qi	Qo	hr.	Qi	Qo
10	0	0	16	1.4	4.8	22	0.7	1.0
11	0.3	0.3	17	1.2	1.8	23	0.7	1.0
12	23.9	6.8	18	1.1	1.5	24	0.6	0.9
13	4.6	7.7	19	1.0	1.4	25	0.0	0.5
14	2.4	7.5	20	0.9	1.3	26	0	0
15	1.6	6.4	21	0.7	1.1			

Inflow and Outflow Hydrograph Values
Qi taken from TR-20 Hydrology Computer Run
Qo taken from Routing Worksheet – Fig. 5-28



Peak 25.9 cfs, 12.13 hrs

Calculations=
$(0.40\ in^2)(10\ cfs/in)(2.5\ hrs/in)(3600\ sec/hr)=35,820\ ft^3$
$=0.82$ ac–ft

c:\g5_29.dwg

5-10 WATER QUALITY CALCULATION PROCEDURES

This section presents procedures for complying with the water quality criterion outlined in the stormwater management regulations. The water quality criterion represent a consolidation of the requirements of three state agencies charged with the responsibility of monitoring and improving the water resources of the Commonwealth: The Department of Conservation and Recreation (DCR), the Department of Environmental Quality (DEQ), and the Chesapeake Bay Local Assistance Department (CBLAD). The specific responsibilities of these agencies are presented in **Chapter 1**.

The stormwater management water quality regulations require compliance by either a **performance-based water quality criteria** or a **technology-based water quality criteria**. The performance-based water quality criteria requires the designer to implement a Best Management Practice (BMP) or combination of BMPs which effectively remove the anticipated increase in pollutant load from a development site. This approach requires the designer to calculate the pollutant load to be removed, implement a BMP strategy, and then calculate the performance of that strategy, based on the effectiveness or pollutant removal efficiency of the selected BMP(s).

The technology-based water quality criteria simply states that for land uses of given amounts of impervious cover, measured in percent, there are best available technologies with which to remove the anticipated pollutant load increase.

These two criterion are considered to be equivalent when implemented as described in this handbook. A more detailed discussion of these water quality criterion and the selection of water quality BMPs is presented in **Chapter 2**.

5-10.1 Performance-Based Water Quality Criteria

This procedure is for determining compliance with the performance-based water quality criteria of the Commonwealth's stormwater management regulations. The **Performance-based water quality criteria** is defined as follows:

For land development, the calculated post-development nonpoint source pollutant runoff load shall be compared to the calculated pre-development load based upon the average land cover condition <u>or</u> the existing site condition. A BMP(s) shall be located, designed, and maintained to achieve the target pollutant removal efficiencies specified in **Table 5-14** and to effectively reduce the pollutant load to the required level based upon the four applicable land development situations for which the performance criteria apply. (Refer to **STEP 3** for a discussion of the development situations.)

The "nonpoint source pollutant runoff load" or "pollutant discharge" is defined as the average amount of a particular pollutant(s) measured in pounds per year, delivered in a diffuse manner by stormwater runoff. The calculation procedure described herein uses the contaminant **phosphorous** for the purposes of calculating pollutant discharge in order to determine compliance with the performance-based water quality criteria. **However, other pollutants may be targeted if**

determined to be more appropriate for the intended land use. Refer to **Chapter 2** for a discussion of urban nonpoint source pollution.

The accepted calculation procedure for the determining the pre- and post-developed pollutant loads from development sites is referred to as the Simple Method. A more detailed discussion and derivation of the Simple Method can be found in *Appendix A* of Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, published by the Metropolitan Washington Council of Governments. The simple method uses impervious cover as the key variable in calculating the levels of pollutant export. (It should be noted that other more data intensive methods for calculating pollutant loads are available. DCR will evaluate the option of utilizing these methods in the future.

Equation 5-14 presents the Simple Method General Pollutant Load Equation.

$$L = P \times P_j \times [0.05 + (0.009 \times I)] \times C \times A \times 2.72 \div 12$$

Equation 5-14
Simple Method Pollutant Load (L)

where:

L = *relative total phosphorous load (pounds per year)*
P = *average annual rainfall depth (inches), assumed to be 43 inches for Virginia**
P_j = *unitless correction factor for storm with no runoff = 0.9*
I = *percent impervious cover (percent expressed in whole numbers)*
C = *flow-weighted mean pollutant concentration = 0.26 milligrams per liter*
A = *applicable area (acres)*
Note: 12 and 2.72 are conversion factors
* - The annual rainfall depth may vary across the commonwealth based on locally collected rainfall data. The designer should verify actual rainfall values which may be required in the local jurisdiction. Also note that the use of the same value in the pre- and post-developed computations allows for the cancellation of this and other values as discussed below.

The purpose of this calculation is to provide a comparison between the pre- and post-development pollutant loads. Therefore, in an effort to simplify **Equation 5-14**, any value which will not change with the development of land, such as rainfall (P) and the flow weighted mean pollutant concentration (C), and any constants, such as the correction factor(P_j) and conversion factors, can be multiplied through. Thus **Equation 5-14** simplifies to:

$$L = [0.05 + (0.009 \times I)] \times A \times 2.28$$

Equation 5-15
Simple Method Pollutant Load (L), Simplified

where: L = *relative total phosphorous load (pounds per year)*
I = *percent impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The Performance-based criteria requires that a pre- and post-developed condition pollutant load be calculated in order to determine the relative increase. A consistent, calculated pre-developed annual load (L_{pre}), or base annual load, with which to compare the calculated post-developed annual load (L_{post}) is therefore required. The Chesapeake Bay Local Assistance Department has determined a base line annual load of phosphorous for Tidewater Virginia and has established a corresponding baseline impervious value, or average land cover condition ($I_{watershed}$), of 16%. A locality may choose to adopt this value as the pre-developed default for the entire locality. **Or** the locality may choose to calculate a watershed or locality-wide pre-developed annual load and corresponding impervious value, and designate a watershed-specific or locality-specific average land cover condition.

Localities have the following options when determining average land cover conditions:

Option 1: A locality may designate specific watersheds within its jurisdiction and calculate the average land cover condition ($I_{watershed}$) and associated average total phosphorous loading for those watersheds (**Table 5-15** presents representative land uses and associated percent impervious cover and phosphorous export values); or

Option 2: A locality may assume the Chesapeake Bay default value for total phosphorous loading of 0.45 pounds/acre/year (F_{VA}) and an equivalent impervious cover ($I_{watershed}$) of 16 percent for its entire jurisdiction.

The calculation of watershed-specific average total phosphorous loadings must be based upon the following:

1. existing land use data at time of local program adoption,
2. watershed size, and
3. determination of equivalent values of impervious cover for non-urban land uses which contribute nonpoint source pollution, such as agriculture, silviculture, etc.

Some localities may begin with *Option 2* while they gather the necessary data for *Option 1*. The average land cover condition, once established for a locality (or watershed), **should not change**, and the designer simply uses that value as the existing condition baseline value for the specific watershed or locality in which the project is located.

5-10.2 Performance-Based Water Quality Calculation Procedure

The following steps represent the performance-based water quality calculation procedure:

STEP 1 **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

STEP 2 **Determine the existing impervious cover ($I_{existing}$) or use the average land cover condition ($I_{watershed}$)**

STEP 3 **Determine the appropriate development situation.**

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

STEP 6 **Determine the relative pollutant removal requirement (RR).**

STEP 7 **Identify best management practice (BMP) options for the site.**

The following discussion presents each step of the calculation procedure:

STEP 1 **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

Applicable Area

The applicable area (A) is the parcel of land being developed. For large developments such as subdivisions, shopping centers, or office / institutional campus style developments, use of the entire parcel or development areas can result in unreasonable water quality requirements. In these cases, the designation of a *planning area* may be more appropriate. A planning area is a designated portion of the parcel of land, measured in acres, on which the development project is located. The planning area may be established by drainage areas or development areas. A designated planning area can be helpful when analyzing developments where the density of impervious cover, construction phasing, or other factors vary across the total site and create distinctly separate areas of analysis. (The concept and advantages of planning areas are discussed further in **Chapter 2**.)

The use of planning areas must be preceeded by the development of a master plan to ensure that the entire development is accounted for, as well as document the consistent application of the designated planning areas (land can not be included in more than one planning area).

Post-development Impervious Cover (I_{post})

The designer must determine the amount of post-development impervious cover (I_{post}), in percent, within the applicable area. The zoning classifications or proposed density of a site will allow the designer to estimate impervious cover. It is important that the roadways, sidewalks, and other public or common ground improvements are included in the overall total impervious cover calculations when calculating the average lot size and the associated impervious cover. Compliance and final engineering calculations, however, should be based on impervious cover shown on the final site or subdivision plan. A locality may set minimum acceptable impervious percentages for particular land uses, and may also require a determination of the actual proposed impervious cover and **use the higher value**. Representative land use categories and associated average impervious cover values are shown in **Table 5-15**.

STEP 2 Determine the existing impervious cover ($I_{existing}$) or use the average land cover condition ($I_{watershed}$) as determined by the locality.

Existing Impervious Cover ($I_{existing}$):

The existing impervious cover ($I_{existing}$) is the percentage of the site that is occupied by impervious cover prior to the development of the proposed project. For new construction there is typically no existing impervious cover and therefore the average land cover condition or the watershed-specific value is used. Two of the four development situations presented in this standard, however, are based on the presence of existing site features or previous development and use the existing impervious cover as the basis for determining the pre-development total phosphorous load (L_{pre}).

Average Land Cover Condition ($I_{watershed}$):

A locality must establish the base pollutant load for specific watersheds or for the locality as a whole based on all of the land uses within the established boundary and, in turn, must determine the corresponding average land cover condition ($I_{watershed}$) measured in percent impervious cover. The average land cover condition, therefore, will be a watershed- or locality-specific value, or the Chesapeake Bay default value of 16%. The average land cover condition, once established for a locality (or watershed), should not change, and the designer simply uses that value as the pre-developed or existing average land cover condition for the specific watershed or locality in which the project is located.

STEP 3 Determine the appropriate development situation.

The performance-based criteria is applied through the use of four development situations. The application of each of these situations uses the same development characteristic (impervious cover) to determine the post-development pollutant load (L_{post}). However, the pre-development pollutant load (L_{pre}) is determined using either the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$), depending on the development situation. The situations are as follows:

Situation 1: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than the average land cover condition ($I_{watershed}$).

Requirement: No reduction in the after development pollutant discharge (L_{post}) is required.

Situation 2: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

Situation 3: Land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$), whichever is greater.

Situation 4: Land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP ($L_{pre(existingBMP)}$). The existing BMP shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. Development Situations 2 through 4 proceed to STEP 4.

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

The pre-developed pollutant load is based on either the average land cover condition ($L_{pre(watershed)}$): Situation 2; or the existing site conditions ($L_{pre(existing)}$): Situation 3; **or** the existing site conditions while being served by a water quality BMP ($L_{pre(existingBMP)}$): Situation 4.

The simplified version of the Simple Method Pollutant Load Equation (**Equation 5-15**) is modified by inserting the specific values of I ($I_{watershed}$ or $I_{existing}$) to calculate the relative pre-development total phosphorous load for the different development situations (2 through 4). The Simple Method Pollutant Load Equation is applied to the development situations as follows:

Situation 2:

The treatment requirement for Situation 2 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$). Therefore, the Simple Method Pollutant Load Equation is slightly modified to calculate the relative pre-development pollutant load ($L_{pre(watershed)}$) as follows:

$$L_{pre(watershed)} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$$

Equation 5-16
Pollutant Load Based on Average Land Cover Conditions ($L_{pre(watershed)}$)

where:

$L_{pre(watershed)}$	=	*relative pre-development total phosphorous load (pounds per year)*
$I_{watershed}$	=	*average land cover condition for specific watershed or locality* ***or*** *the Chesapeake Bay default value of 16% (percent expressed in whole numbers)*
A	=	*applicable area (acres)*

Situation 3:

The treatment requirement for Situation 3 states that the pollutant discharge after development (L_{post}) shall not exceed the greater of: 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

The pre-development pollutant discharge must be calculated twice in order to determine compliance with this requirement: first based on the existing impervious cover ($I_{existing}$) to calculate the pre-development load ($L_{pre(existing)}$) (**Equation 5-17**); and again based on the average land cover condition ($I_{watershed}$) to calculate the pre-development load ($L_{pre(watershed)}$) (**Equation 5-16**). The Simple Method Pollutant Load Equation is used as follows:

$$L_{pre(existing)} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$$

Equation 5-17
Pollutant Load Based on Existing Site Conditions ($L_{pre(existing)}$)

where: $L_{pre(existing)}$ = *relative pre-development total phosphorous load (pounds per year)*
$I_{existing}$ = *existing site impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$) is calculated the same as was done in **STEP 2** using **Equation 5-16**. The comparison of $L_{pre(existing)}$ less 10% and $L_{pre(watershed)}$ is made in **STEP 5** of this procedure.

Situation 4:

The requirement for Situation 4 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP(s) ($L_{pre(existingBMP)}$). The existing BMP(s) shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

This requirement assumes that either all or a portion of the pollutant load generated by the existing impervious cover on a development is being reduced by one or more BMPs designed and constructed for that purpose. It becomes the responsibility of the designer or applicant to demonstrate that the facility was designed and constructed in accordance with the proper design standards and specifications, and is in proper functioning condition in order to justify the pollutant removal efficiency attributed to that particular BMP. Acceptable pollutant removal efficiency values attributed to some of the more commonly used BMPs for which there is adequate performance data are presented in **Table 5-14**. **Chapter 3** provides the design and maintenance requirements for these BMPs.

It should be noted that there may be more than one existing BMP. The drainage area to each BMP must be evaluated independantly. All areas being evaluated should be clearly documented on an existing conditon drainage area map.

The pre-developed total phosphorous load based on existing site conditions ($L_{pre(existing)}$) is calculated using **Equation 5-17**. The designer must then determine how much of the existing impervious cover is captured by the existing BMP(s), and the relative pollutant load removed. The Simple Method Pollutant Load Equation is therefore applied independently to each BMP drainage area of the site to determine the relative pollutant load of the area draining to the existing BMP(s) (**Equation 5-18**) and then the efficiency of each BMP is applied to the respective load to determine the load removed (**Equation 5-19**) as follows:

$$L_{pre(BMP)} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28$$

Equation 5-18
Pollutant Load to Existing BMP ($L_{pre(BMP)}$)

where:
$L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP (pounds per year)*
$I_{pre(BMP)}$ = *existing impervious cover to existing BMP (percent expressed in whole numbers)*
$A_{existBMP}$ = *drainage area to existing BMP (acres)*

The relative pollutant load removed by the existing BMP ($L_{removed(existingBMP)}$) is determined as follows:

$$L_{removed(existingBMP)} = L_{pre(BMP)} \times EFF_{existBMP}$$

Equation 5-19
Pollutant Load Removed by Existing BMP ($L_{removed(existingBMP)}$)

where:
$L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP (pounds per year)*
$L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)*
$EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*

Equations 5-18 and **5-19** are thus applied independantly to each existing BMP on the site.

The relative pre-development pollutant load from the site can now be calculated using **Equation 5-20** as follows:

$$L_{pre(existingBMP)} = L_{pre(existing)} \quad (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20
Pollutant Load Based on Existing BMP Removal Efficiency ($L_{pre(existingBMP)}$)

where:
$L_{pre(existingBMP)}$ = *relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)*
$L_{pre(existing)}$ = *relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)*
$EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*
$L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)*

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

The post-development pollutant load (L_{post}) is calculated based on the proposed impervious cover for each development situation. The Simple Method Pollutant Load Equation based on the proposed post-development impervious cover (I_{post}) is used as follows:

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$

Equation 5-21
Pollutant Load Based on Post-Development Site Conditions (L_{post})

where:
- L_{post} = *relative post-development total phosphorous load (pounds per year)*
- I_{post} = *post-development impervious cover (percent expressed in whole numbers)*
- A = *applicable area (acres)*

STEP 6 **Determine the relative pollutant removal requirement (RR).**

The pollutant removal requirement (RR) is defined as the relative amount of the keystone pollutant (in pounds per year) which must be removed by a BMP. The development situations discussed in **STEP 3** present the different removal or treatment requirements for each situation. There is no treatment requirement for Situation 1 due to the low density of development (proposed impervious cover less than the average land cover condition). The requirements for Situations 2, 3, and 4 are as follows:

Situation 2: $RR = L_{post} \quad L_{pre(watershed)}$

Situation 3: $RR = L_{post} \quad (0.9 \times L_{pre(existing)})$; **or**
$RR = L_{post} \quad L_{pre(watershed)}$, which ever value of RR is less.

Situation 4: $RR = L_{post} \quad L_{pre(existingBMP)}$

If the calculated RR value is less than or equal to zero, no BMPs are required. If the RR value greater than zero, continue on with STEP 7.

STEP 7 **Identify best management practice (BMP) options for the site.**

The selection criteria for choosing an appropriate BMP for any given development site is often dictated by the physical characteristics of the site, such as soil types, topography, and drainage area. In addition, the pollutant removal requirement (RR) for the site may dictate that a BMP with a high removal efficiency (EFF_{BMP}) be used, while the physical characteristics of the site may dictate that a combination of strategically located BMPs be used. Specific siting and design criterion, as well as the accepted pollutant removal efficiencies for generally acceptable BMPs, are discussed in **Chapter 3: BMP Minimum Standards**.

The first step in determining which BMP may satisfy the pollutant removal requirement is to determine the necessary BMP pollutant removal efficiency. When the entire development is to be served by one BMP, this can be calculated using the following equation:

$$EFF = (RR \div L_{post}) \times 100$$

Equation 5-22
Required Pollutant Removal Efficiency (EFF)

where:
- EFF = *required pollutant removal efficiency*
- RR = *pollutant removal requirement (pounds per year)*
- L_{post} = *relative post-development total phosphorous load, Equation 5-21 (pounds per year)*

If more than one BMP will be used on the site, the removal requirement (RR) and post-development total load (L_{post}) must be calculated for each area using **Equation 5-22**. The designer can then use the required pollutant removal efficiency (RR) value to make a preliminary BMP(s) selection from **Table 5-15**. This is a preliminary selection since the specific siting and design criteria for the selected BMP must now be satisfied. Refer to **Chapter 3** for more information.

Once the BMP is selected and sited the designer must verify that the BMP(s) satisfies the removal requirement (RR) for the development. This is done by applying the pollutant removal efficiency (EFF_{BMP}) of the selected BMP to the post-developed pollutant load **entering the BMP as sited** (L_{BMP}). If the entire site drains to the proposed BMP, then the post-development pollutant load entering the BMP (L_{BMP}) is that which was calculated in **STEP 5** ($L_{post} = L_{BMP}$). In many cases, however, the topographic constraints of the site, or siting constraints of the specific BMP chosen, may result in some impervious areas not draining to the proposed BMP. Therefore, the Simple Method General Pollutant Load Equation must be applied to the actual drainage area of the BMP(s) as follows:

$$L_{BMP} = [0.05 + (0.009 \times I_{BMP})] \times A_{propBMP} \times 2.28$$

Equation 5-23
Pollutant Load Entering Proposed BMP (L_{BMP})

where:
- L_{BMP} = *relative post-development total phosphorous load entering proposed BMP(pounds per year)*
- I_{BMP} = *post-development percent impervious cover to proposed BMP (percent expressed in whole numbers)*
- $A_{propBMP}$ = *drainage area to proposed BMP (acres)*

The load removed by the BMP is then calculated as follows:

$$L_{removed} = Eff_{BMP} \times L_{BMP}$$

Equation 5-24
Pollutant Load Removed by Proposed BMP ($L_{removed}$)

where:
$L_{removed}$ = *post-development total phosphorous load removed by proposed BMP (pounds per year)*
Eff_{BMP} = *pollutant removal efficiency of BMP (expressed in decimal form)*
L_{BMP} = *relative post-development total phosphorous load entering proposed BMP,* ***Equation 5-23*** *(pounds per year)*

The calculation in this step is performed for each BMP and the various $L_{removed}$ values for the existing and proposed BMPs are summed for the total pollutant load removal as follows:

$$L_{removed/total} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots + L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)}$$

Equation 5-25
Total Pollutant Load Removed by Proposed BMPs ($L_{removed/total}$)

where:
$L_{removed/total}$ = ***total*** *pollutant load removed by proposed BMPs (pounds per year)*
$L_{removed/BMP1}$ = *pollutant load removed by proposed BMP No. 1,* ***Equation 5-24***
$L_{removed/BMP2}$ = *pollutant load removed by proposed BMP No. 2,* ***Equation 5-24***
$L_{removed/BMP3}$ = *pollutant load removed by proposed BMP No. 3,* ***Equation 5-24***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 1,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 2,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 3,* ***Equation 5-19***

The BMP or combination of BMPs is determined to be adequate if the total pollutant load removed ($L_{removed/total}$) is greater than or equal to the removal requirement (RR) calculated in **STEP 6**: $L_{removed/total}$ RR

If the total load removed is less than the removal requirement (RR) than an alternate BMP or combination of BMPs must be selected. It may be possible to simply increase the drainage area to the BMP(s) (if the entire site does not already drain to the BMP) in order to increase the overall pollutant removal from the site. Another option may be to reduce the impervious cover of the development in order to lower the removal requirement. The designer may also investigate the opportunities to capture off-site impervious area drainage in the proposed BMP to compensate for on-site areas which cannot be captured. In all cases the designer should contact the local program authority to determine if options are available in the local program as a result of a watershed or regional BMP plan.

Table 5-14
Water Quality BMP Pollutant Removal Efficiencies

Water Quality BMP*	Target Pollutant Removal Efficiency	Percent Impervious Cover
Vegetated filter strip	10%	16-21%
Grassed swale	15%	
Constructed wetlands	30%	
Extended detention (2 x WQ Vol)	35%	22 -37%
Retention basin I (3 x WQ Vol)	40%	
Bioretention basin	50%	
Bioretention filter	50%	
Extended detention-enhanced	50%	38 -66%
Retention basin II (4 x WQ Vol)	50%	
Infiltration (1 x WQ Vol)	50%	
Sand filter	65%	
Infiltration (2 x WQ Vol)	65%	67 -100%
Retention basin III (4 x WQ Vol with aquatic bench)	65%	

* Innovative or alternative BMPs not included in this table may be allowed at the discretion of the local program administrator, the plan approving authority, or the Department

Table 5-15
Simple Method General Pollutant Load Equation Solved for Incremental Impervious Cover Values (Urban Land Uses)

Representative Land Uses	Average Impervious Cover	Annual Pollutant Load (lb/ac/yr)
	0	0.11
2-5 Acre Residential	5	0.22
	10	0.32
	15	0.42
1 Acre Residential	20	0.52
½ Acre Residential	25	0.63
1/3 Acre Residential	30	0.73
1/4 Acre Residential	35	0.83
	40	0.94
1/8 Acre Residential	45	1.04
	50	1.14
Townhouses/ Garden Apartments	55	1.24
	60	1.35
	65	1.45
Light Industrial	70	1.55
	75	1.65
Heavy Industrial/ Commercial	80	1.76
	85	1.86
	90	1.96
	95	2.06
Pavement	100	2.17

Note: The average impervious cover values may be used for estimating or planning purposes when considering the representative land use as shown. When possible, final design calculations should be based on actual percent impervious cover as measured from the site plan.

Table 5-15 (Cont.)
Simple Method General Pollutant Load Equation Solved for Incremental Impervious Cover Values
(Non-Urban Land Uses)
(in pounds/acre/year)

Land Use	Silt Loam Soils	Loam Soils	Sandy Loam Soils
Conventional Tillage Cropland	3.71	2.42	0.83
Conservation Tillage Cropland	2.32	1.52	0.52
Pasture Land	0.91	0.59	0.20
Forest Land	0.19	0.12	0.04

CHAPTER 5

APPENDIX

APPENDIX 5A

a   b   Constants for Virginia

a b Constants for Virginia

		2 YEAR		10 YEAR		100 YEAR	
COUNTY	#	a	b	a	b	a	b
ARLINGTON	00	119.34	17.86	178.78	20.66	267.54	22.32
ACCOMACK	01	107.75	14.69	175.90	20.64	277.44	24.82
ALBEMARLE	02	106.02	15.51	161.60	18.73	244.82	20.81
ALLEGHENY	03	95.47	13.98	145.89	17.27	220.94	19.29
AMELIA	04	112.68	15.11	173.16	18.81	266.77	22.13
AMHERST	05	106.72	15.39	162.75	18.83	245.52	21.02
APPOMATTOX	06	109.11	15.39	167.44	19.12	254.03	21.61
AUGUSTA	07	84.21	10.44	135.74	14.54	210.02	16.99
BEDFORD	09	114.59	17.21	171.51	20.47	258.17	22.80
BLAND	10	105.33	16.56	162.75	20.41	247.84	22.87
BOTETOURT	11	110.32	16.95	164.94	20.01	247.92	22.16
BRUNSWICK	12	126.74	17.27	190.73	21.52	287.02	24.46
BUCHANAN	13	87.14	13.22	128.51	15.15	189.98	16.22
BUCKINGHAM	14	109.95	15.41	168.28	19.11	254.59	21.47
CAMPBELL	15	110.26	15.76.	167.27	19.18	252.65	21.56
CAROLINE	16	121.21	17.33	182.56	20.88	275.65	23.30
CARROLL	17	119.79	18.65	188.13	23.81	288.94	27.06
CHARLES CITY	18	124.23	17.14	186.52	21.05	281.04	23.85
CHARLOTTE	19	109.87	14.71	171.75	19.25	265.18	22.56
CHESTERFIELD	20	124.66	17.55	186.15	21.03	277.94	23.26
CLARKE	21	94.13	12.88	141.03	15.39	210.66	16.85
CRAIG	22	106.67	16.54	166.19	20.94	251.27	22.95
CULPEPER	23	111.90	16.25	169.78	19.51	255.26	21.52
CUMBERLAND	24	111.34	15.29	172.73	19.29	271.55	24.02

COUNTY	#	a	b	a	b	a	b
DICKENSON	25	87.03	13.10	128.09	14.82	190.08	15.98
DINWIDDIE	26	125.08	17.29	189.77	21.51	284.68	24.02
ESSEX	28	119.70	16.76	180.50	20.18	271.79	22.58
FAIRFAX	29	117.06	17.34	178.32	20.49	269.23	22.40
FAUQUIER	30	116.55	17.52	172.47	20.02	255.06	21.38
FLOYD	31	121.22	19.16	185.59	23.38	281.91	26.26
FREDERICK	34	93.79	13.15	141.02	15.77	211.40	17.42
GILES	35	106.14	16.72	165.04	20.80	252.79	23.46
GLOUCESTER	36	119.62	16.09	182.54	20.40	276.43	23.35
GOOCHLAND	37	114.42	15.95	177.24	19.93	269.07	22.27
GRAYSON	38	119.29	18.94	176.02	22.06	262.24	24.25
GREEN	39	105.71	15.10	159.92	18.20	241.18	20.34
GREENSVILLE	40	129.97	17.80	194.08	22.01	291.37	24.83
HALIFAX	41	111.92	15.14	173.81	19.52	267.09	22.70
HANOVER	42	122.80	17.29	185.01	20.91	278.40	23.40
HENRICO	43	123.51	17.35	185.51	21.13	277.61	23.44
HENRY	44	116.19	17.33	177.84	21.34	270.32	24.01
HIGHLAND	45	90.13	12.61	134.38	15.02	199.74	16.50
ISLE OF WIGHT	46	125.69	17.02	190.34	21.71	287.14	24.73
JAMES CITY	47	121.86	16.58	185.06	20.81	279.14	23.67
KING GEORGE	48	120.31	17.28	181.05	20.50	273.29	22.83
KING & QUEEN	49	113.84	15.29	179.09	19.95	275.98	23.15
KING WILLIAM	50	114.92	15.58	180.36	20.13	277.03	23.26
LANCASTER	51	109.80	14.49	170.27	18.72	259.78	21.41
LEE	52	93.78	14.40	143.28	17.58	215.10	19.22
LOUDOUN	53	104.05	14.91	157.67	17.71	237.83	19.65

COUNTY	#	a	b	a	b	a	b
LOUISA	54	112.63	15.89	174.35	19.72	265.20	22.11
LUNENBERG	55	122.01	16.82	184.70	20.80	278.38	23.48
MADISON	56	106.87	15.33	161.43	18.49	242.78	20.62
MATHEWS	57	118.61	15.83	180.56	20.17	274.12	23.29
MECKLENBERG	58	121.77	16.55	184.54	20.74	278.33	23.48
MIDDLESEX	59	110.72	14.57	172.76	19.15	264.49	22.13
MONTGOMERY	60	118.78	19.21	176.95	22.39	262.93	24.17
NELSON	62	103.46	14.52	160.23	18.36	245.04	20.89
NEW KENT	63	121.03	16.58	183.93	20.72	277.89	23.51
NORFOLK	64	124.88	17.02	190.64	22.14	288.73	25.60
NORTHAMPTON	65	111.07	14.78	173.72	19.63	267.48	23.04
NORTHUMBERLAND	66	111.20	14.99	171.55	19.00	260.59	21.63
NOTTOWAY	67	122.38	17.06	183.97	20.87	275.78	23.19
ORANGE	68	116.77	16.63	178.14	20.19	270.55	22.72
PAGE	69	84.19	10.29	135.43	14.29	209.57	16.86
PATRICK	70	123.68	19.26	189.08	23.60	284.78	26.12
PITTSYLVANIA	71	112.30	16.02	173.58	20.27	263.51	22.98
POWHATAN	72	114.14	15.64	175.93	19.65	266.86	22.15
PRINCE EDWARD	73	111.01	15.06	172.73	19.29	264.28	22.20
PRINCE GEORGE	74	126.22	17.46	188.62	21.39	283.12	24.09
VIRGINIA BEACH	75	129.20	17.84	196.25	22.74	294.74	26.33
PRINCE WILLIAM	76	116.04	17.08	176.18	20.19	266.75	22.36
PULASKI	77	117.44	18.71	182.33	23.39	279.39	26.49
RAPPAHANNOCK	78	104.86	15.05	159.40	18.34	239.30	20.19
RICHMOND	79	117.41	16.23	177.35	19.85	267.20	22.24
ROANOKE	80	117.53	18.79	174.97	21.80	261.95	23.81

COUNTY	#	a	b	a	b	a	b
ROCKBRIDGE	81	84.23	10.46	143.41	15.89	229.43	19.56
ROCKINGHAM	82	83.83	10.55	128.80	13.37	195.24	15.29
RUSSELL	83	92.64	14.17	143.00	17.32	216.40	19.36
SCOTT	84	92.64	14.17	143.00	17.32	216.40	19.35
SMYTH	86	106.19	16.57	169.30	21.37	262.49	24.57
SOUTHAMPTON	87	129.91	17.77	195.84	22.34	294.40	25.43
SPOTSYLVANIA	88	117.31	16.86	179.21	20.48	269.84	22.55
STAFFORD	89	118.72	17.34	179.62	20.64	270.74	22.79
SURRY	90	124.79	16.97	188.62	21.39	283.36	24.16
SUSSEX	91	130.37	18.03	193.23	21.91	287.99	24.56
TAZEWELL	92	91.25	13.56	141.61	17.04	217.59	19.48
WARREN	93	89.03	11.53	137.69	14.73	210.46	16.87
WASHINGTON	95	106.65	16.86	162.19	20.02	244.60	21.98
WESTMORELAND	96	114.40	15.76	174.96	19.47	266.16	22.12
WISE	97	89.83	13.49	132.05	15.44	194.10	16.35
WYTHE	98	116.78	18.83	174.91	22.13	261.68	24.25
YORK	99	122.93	16.72	186.78	21.22	282.80	24.39

		2 YEAR		10 YEAR		100 YEAR	
CITIES	**#'s**	**a**	**b**	**a**	**b**	**a**	**b**
RICHMOND	**127/43**	122.47	17.10	185.51	21.13	278.85	23.60
HAMPTON	**114/27**	123.93	16.94	186.78	21.22	283.18	24.56
LYNCHBURG	**118/15**	107.39	15.15	166.87	19.37	255.02	22.08
SUFFOLK	**133/61**	129.97	17.80	196.63	22.61	298.69	26.35
NEWPORT NEWS	**121/94**	126.11	17.37	189.27	21.62	285.24	24.71

Source: Virginia Department of Transportation

APPENDIX 5B

Filter and Drainage Diaphragm Design

USDA-SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters

USDA-SCS Soil Mechanics Note No. 3: Soil Mechanics Considerations for Embankment Drains

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

Part 633
National
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Handbook

Chapter 26 Gradation Design of Sand and Gravel Filters

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Preface

Most of the criteria in this document was originally issued in Soil Mechanics Note 1, revised January 1986. This revision of Soil Mechanics Note 1 and any future revisions of other Soil Mechanics Notes will be placed in the National Engineering Handbook, Part 633, Soil Engineering. This material is Chapter 26, Gradation Design of Sand and Gravel Filters.

Acknowledgments

The criteria in this document are based on the results of an extensive laboratory filter study carried out by the Soil Conservation Service (SCS) at the Soil Mechanics Laboratory in Lincoln, Nebraska, from 1980 to 1985. The principals involved in this study were **Lorn P. Dunnigan**, SCS (retired), **James R. Talbot**, SCS (retired), and **James L. Sherard**, consultant (deceased).

Revisions were developed in 1993 by **Danny K. McCook**, assistant head, Soil Mechanics Laboratory, SCS, Fort Worth, Texas; **Charles H. McElroy**, head of the Soil Mechanics Laboratory, SCS, Fort Worth, Texas; and **James R. Talbot**, national soils engineer, SCS, Washington, DC (retired). Danny McCook developed the example problems.

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Chapter 26 Gradation Design of Sand and Gravel Filters

Contents:

Figures

Examples

Chapter 26 Gradation Design of Sand and Gravel Filters

633.2600 Purpose

Chapter 26 presents criteria for determining the grain-size distribution (gradation) of sand and gravel filters needed to prevent internal erosion or piping of soil in embankments or foundations of hydraulic structures.

These criteria are based on results of an extensive laboratory filter study carried out by the Soil Conservation Service at the Soil Mechanics Laboratory in Lincoln, Nebraska, from 1980 to 1985. (See Section 633.2605, References, for published reports.)

Refer to section 633.2604 for definitions used in this chapter.

633.2601 Basic purpose of filters and drains

Filters are placed in embankment zones, foundations, or other areas of hydraulic structures for two purposes:

- To intercept water flowing through cracks or openings in a base soil and block the movement of eroding soil particles into the filter. Soil particles are caught at the filter face, reducing the flow of water through cracks or openings and preventing further erosion and enlargement of the cracks or openings.
- To intercept water flowing through the pores of the base soil, allowing passage of the water while preventing movement of base soil particles. Without filters, piping of susceptible base soils can occur when seepage gradients or pressures are high enough to produce erosive discharge velocities in the base soil. The filter zone is generally placed upstream of the discharge point where sufficient confinement prevents uplift or blow-out of the filter.

Drains consist of sand, gravel, or a sand and gravel mixture placed in embankments, foundations, and backfill of hydraulic structures, or in other locations to reduce seepage pressure. A drain's most important design feature is its capacity to collect and carry water to a safe outlet at a low gradient or without pressure build-up. Drains are often used downstream of or in addition to a filter to provide outlet capacity.

Combined filters and drains are commonly used. The filter is designed to function as a filter and as a drain.

633.2602 Permeability and capacity

The laboratory filter study clearly demonstrated that graded filters designed in accordance with these criteria will seal a crack. The sealing begins when water flows through a crack or opening and carries soil particles eroded from the sides of the openings. Eroding soil particles collect on the face of the filter and seal the crack at the interface. Any subsequent flow is through the pores of the soil. If filters are designed to intercept cracks, the permeability required in the filter zone should be based on the steady state seepage flow through the pores of the base soil alone. The hydraulic capacity of any cracks need not be considered in designing the filter because the cracks have been shown to seal.

Where saturated steady-state seepage flow will not develop, for instance in dry dams for flood control having a normal drawdown time of 10 days or less, filter capacity need only be nominal. Filters designed either to protect against steady state seepage or internal erosion through cracks are to be thick enough to compensate for potential segregation and contamination of the filter zones during construction. They must also be thick enough that cracks cannot extend through the filter zone during any possible differential movements.

A zone of coarser materials immediately downstream or below the filter, or both, provides additional capacity to collect and convey seepage to a controlled outlet. In some cases a strip drain is used, and in others a perforated collector pipe is employed to outlet the collected seepage. To prevent movement of the filter materials into the coarse drain materials, the coarse drain materials must be designed for the proper gradation using procedures in this subchapter. Perforations in collector pipes must also be sized properly to prevent movement of the coarse drain materials into the perforations.

633.2603 Determining filter gradation limits

Determine filter gradation limits using the following steps:

Step 1: Plot the gradation curve (grain-size distribution) of the base soil material. Use enough samples to define the range of grain sizes for the base soil or soils. Design the filter using the base soil that requires the smallest D_{15} size for filtering purposes. Base the design for drainage purposes on the base soil that has the largest D_{15} size.

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than No. 4 sieve).

Step 3: Prepare adjusted gradation curves for base soils that have particles larger than the No. 4 (4.75 mm) sieve.

- Obtain a correction factor by dividing 100 by the percent passing the No. 4 (4.75 mm) sieve.
- Multiply the percentage passing each sieve size of the base soil smaller than No. 4 (4.75 mm) sieve by the correction factor determined above.
- Plot these adjusted percentages to obtain a new gradation curve.
- Use the adjusted curve to determine the percentage passing the No. 200 (0.075 mm) sieve in step 4.

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter in accordance with the table 26–2.

If desired, the maximum D_{15} may be adjusted for certain noncritical uses of filters where significant hydraulic gradients are not predicted, such as bedding beneath riprap and concrete slabs. For fine clay base soil that has d_{85} sizes between 0.03 and 0.1 mm, a maximum D_{15} of ≤ 0.5 mm is still conservative. For fine-grained silt that has low sand content, plotting below the "A" line, a maximum D_{15} of 0.3 mm may be used.

Step 6: If permeability is a requirement (see section 633.2602)**, determine the minimum allowable D_{15} in accordance with table 26–3.** Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percentage passing of 60 or less. Criteria are summarized in table 26–4.

Table 26–1 Regraded gradation curve data

Base soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

Table 26–2 Filtering criteria — Maximum D_{15}

Base soil category	Filtering criteria
1	$\leq 9 \times d_{85}$ but not less than 0.2 mm
2	≤ 0.7 mm
3	$\leq \left(\frac{40-A}{40-15}\right)\left[\left(4 \times d_{85}\right) - 0.7\text{mm}\right] + 0.7\text{mm}$ A = % passing #200 sieve after regrading (If 4 x d_{85} is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

This step is required to avoid the use of gap-graded filters. The use of a broad range of particle sizes to specify a filter gradation could result in allowing the use of gap-graded (skip-graded) materials. These materials have a grain size distribution curve with sharp breaks or other undesirable characteristics. Materials that have a broad range of particle sizes may also be susceptible to segregation during placement. The requirements of step 9 should prevent segregation, but other steps are needed to eliminate the use of any gap-graded filters.

Gap-graded materials generally can be recognized by simply looking at their grain size distribution curve. However, for specification purposes, more precise controls are needed. In designing an acceptable filter band using the preliminary control points obtained in steps 1 through 6, the following additional requirements should be followed to decrease the probability of using a gap-graded filter.

Table 26–3 Permeability criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

Table 26–4 Other filter design criteria

Design element	Criteria
To prevent gap-graded filters	The width of the designed filter band should be such that the ratio of the maximum diameter to the minimum diameter at any given percent passing value ≤ 60% is ≤ 5.
Filter band limits	Coarse and fine limits of a filter band should each have a coefficient of uniformity of 6 or less.

First, calculate the ratio of the maximum D_{15} to the minimum D_{15} sizes determined in steps 5 and 6. If this ratio is greater than 5, adjust the values of these control points so that the ratio of the maximum D_{15} to the minimum D_{15} is no greater than 5. If the ratio is 5 or less, no adjustments are necessary. Label the maximum D_{15} size as Control point 1 and the minimum D_{15} size as Control point 2. Proceed to step 8.

The decision on where to locate the final D_{15} sizes within the range established with previous criteria should be based on one of the following considerations:

1. Locate the design filter band at the maximum D_{15} side of the range if the filter will be required to transmit large quantities of water (serve as a drain as well as a filter). With the maximum D_{15} size as the control point, establish a new minimum D_{15} size by dividing the maximum D_{15} size by 5, and locate a new minimum D_{15} size. Label the maximum D_{15} size Control point 1 and the minimum D_{15} size Control point 2.

2. Locate the band at the minimum D_{15} side of the range if it is probable there are finer base materials than those sampled and filtering is the most important function of the zone. With the minimum D_{15} size as the control point, establish a new maximum D_{15} size by multiplying the minimum D_{15} size by 5, and locate a new maximum D_{15} size. Label the maximum D_{15} size Control point 1 and the minimum D_{15} size Control point 2.

3. The most important consideration may be to locate the maximum and minimum D_{15} sizes, within the acceptable range of sizes determined in steps 5 and 6, so that a standard gradation available from a commercial source or other gradations from a natural source near the site would fall within the limits. Locate a new maximum D_{15} and minimum D_{15} within the permissible range to coincide with the readily available material. Ensure that the ratio of these sizes is 5 or less. Label the maximum D_{15} size Control point 1 and the minimum D_{15} size Control point 2.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

Other filter design criteria in step 8

To prevent gap-graded filters—Both sides of the design filter band will have a coefficient of uniformity, defined as:

$$CU = \frac{D_{60}}{D_{10}} \le 6$$

Initial design filter bands by this step will have CU values of 6. For final design, filter bands may be adjusted to a steeper configuration, with CU values less than 6, if needed. This is acceptable so long as other filter and permeability criteria are satisfied.

Calculate a maximum D_{10} value equal to the maximum D_{15} size divided by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.) Calculate the maximum permissible D_{60} size by multiplying the maximum D_{10} value by 6. Label this Control point 3.

Determine the minimum allowable D_{60} size for the fine side of the band by dividing the determined maximum D_{60} size by 5. Label this Control point 4.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5. Label as Control points 5 and 6, respectively.

Table 26–5 Maximum and minimum particle size criteria*

Base soil category	Maximum D_{100}	Minimum D_5, mm
All categories	≤ 3 inches (75 mm)	0.075 mm (No. 200 sieve)

* The minus No. 40 (.425 mm) material for all filters must be nonplastic as determined in accordance with ASTM D4318.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.) Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Sand filters that have a D_{90} less than about 20 mm generally do not require special adjustments for the broadness of the filter band. For coarser filters and gravel zones that serve both as filters and drains, the ratio of D_{90}/D_{10} should decrease rapidly with increasing D_{10} sizes.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band. This results in a preliminary design for a filter band. Complete the design by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than shown in table 26–7. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

Additional design considerations: Note that these steps provide a filter band design that is as well graded as possible and still meets criteria. This generally provides the most desirable filter characteristics. However, in some cases a more poorly graded filter band may be preferable; for example, if more readily available standard gradations are needed or where onsite filters are used for economy.

The design filter band obtained in steps 1 through 12 may be adjusted to a steeper configuration in such cases. The width of the filter band should be maintained so that the ratio of the maximum diameters to the minimum diameters at a given percent finer is no greater than 5 below the 60 percent finer value.

Only the portion of the design filter band above the previously established minimum and maximum D_{15} sizes should be adjusted. The design band may be adjusted so that the coefficients of uniformity of both the coarse and fine sides of the design band are less than 6, but not less than 2, to prevent use of very poorly graded filters.

Table 26–6 Segregation criteria

Base soil category	If D_{10} is :	Then maximum D_{90} is:
	(mm)	(mm)
All categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Table 26–7 Criteria for filters used adjacent to perforated collector pipe

Noncritical drains where surging or gradient reversal is not anticipated	The filter D_{85} must be greater than or equal to the perforation size
Critical drains where surging or gradient reversal is anticipated	The filter D_{15} must be greater than or equal to the perforation size.

Note that the requirements for coefficient of uniformity apply only to the coarse and fine limits of the design filter band. It is possible that an individual, acceptable filter whose gradation plots completely within the specified limits could have a coefficient of uniformity greater than 6 and still be perfectly acceptable. The design steps of this procedure will prevent acceptance of gap-graded filters, which is the main concern associated with filters having a high coefficient of uniformity, and it is not necessary to closely examine the coefficient of uniformity of a particular filter as long as it plots within the design filter band.

Illustrations of these filter design steps are in the following examples. The steps in the filter design process are summarized in appendix 26A. The summary is useful to follow as the example problems are reviewed.

Example 26–1 Fine clay base soil—Category 1

Given: The most important function of the filter being designed is to act as a filter.

Step 1: Plot the gradation curve of the base soil material.

Refer to figure 26–1 for the plotted grain size distribution curve for this example clay base soil, labeled Base soil. The plotted curve is from the following data:

Sieve size	% passing
No 10	100
No. 200	90
0.05 mm	80
0.02 mm	60
0.005 mm	40
0.002 mm	32

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

The example base soil has 100 percent finer than the No. 4 sieve, and the grain size distribution curve does not need to be regraded. Proceed to step 4.

Step 3: Not applicable because the base soil contains no particles larger than the No. 4 sieve

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

The example soil has 90 percent finer than the No. 200 sieve. From table 26–1, the soil is in category 1.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

The filtering criteria for base soil category 1 is (table 26–2): The maximum D_{15} of the filter will be less than or equal to 9 times the d_{85} of the base soil, but not less than 0.2 mm.

The d_{85} size of the base soil is 0.06 mm. Thus, the maximum D_{15} of the filter is

$$\leq 9 \times 0.06 = 0.54 \text{ mm (not} < 0.2 \text{ mm)}$$

This is labeled as Maximum D_{15} in figure 26–1.

Step 6: If permeability is a requirement (section 633.2602), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter will have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but will not be less than 0.1 mm in any case.

The example 26–1 base soil does not have a meaningful d_{15} size. The data show that the base soil has 32 percent finer than 0.002 mm, the smallest commonly determined particle size. Therefore, use the default value of 0.1 mm for the minimum D_{15} of the filter. This value is the preliminary value for minimum D_{15}. Proceed to step 7 for any needed adjustments.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in previous steps 5 and 6 so that the ratio is 5 or less, at any given percent passing of 60 or less. Adjustments may be required based on the following considerations.

For example 26–1, the ratio of the maximum D_{15} to the minimum D_{15} sizes is equal to 0.54 / 0.1 = 5.4. Because the value is slightly greater than 5, a slight adjustment is needed in this step. The minimum D_{15} is the control because filtering is stated as the most important purpose. Label this as Control point 2. Determine an adjusted maximum D_{15} size for the final design filter band as equal to the minimum D_{15} size, 0.10 x 5 = 0.50 mm. This is the final Control point 1 labeled in figure 26–1. Go to step 8.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent using possibly gap-graded filters. Adjust the limits of the design filter band so that coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. Width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

For example 26–1, calculate a value for maximum D_{10} by dividing the maximum D_{15} size of 0.5 mm (determined in step 7) by 1.2 = 0.42 mm. Determine the value for the maximum D_{60} size by multiplying the value of D_{10} by 6 = 0.42 x 6 = 2.5 mm. Label this as Control point 3.

Determine the minimum allowable D_{60} size for the fine side of the band by dividing the determined maximum D_{60} size by 5:

$$\frac{D_{60}}{5} = \frac{2.5}{5} = 0.50$$

Label this Control point 4.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–1.

It also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–1.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Calculate the minimum D_{10} size of the preliminary filter band as equal to the minimum D_{15} value of 0.1 mm (obtained in step 6) divided by 1.2:

$$0.10 / 1.2 = 0.083 \text{ mm}$$

Table 26–6 lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Because the D_{10} value is less than 0.5 mm, the maximum D_{90} size is 20 mm. Label this value as Control point 7 in figure 26–1.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a partial design for the coarse side of the filter band.

Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

Refer to figure 26–1 for an illustration of the complete filter design. Note that adjustments have been made in straight line portions of the design band to intercept even values for percent passing at standard sieve sizes and to prevent the use of very broadly graded filters. The final design specified gradation is shown in table 26–8.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

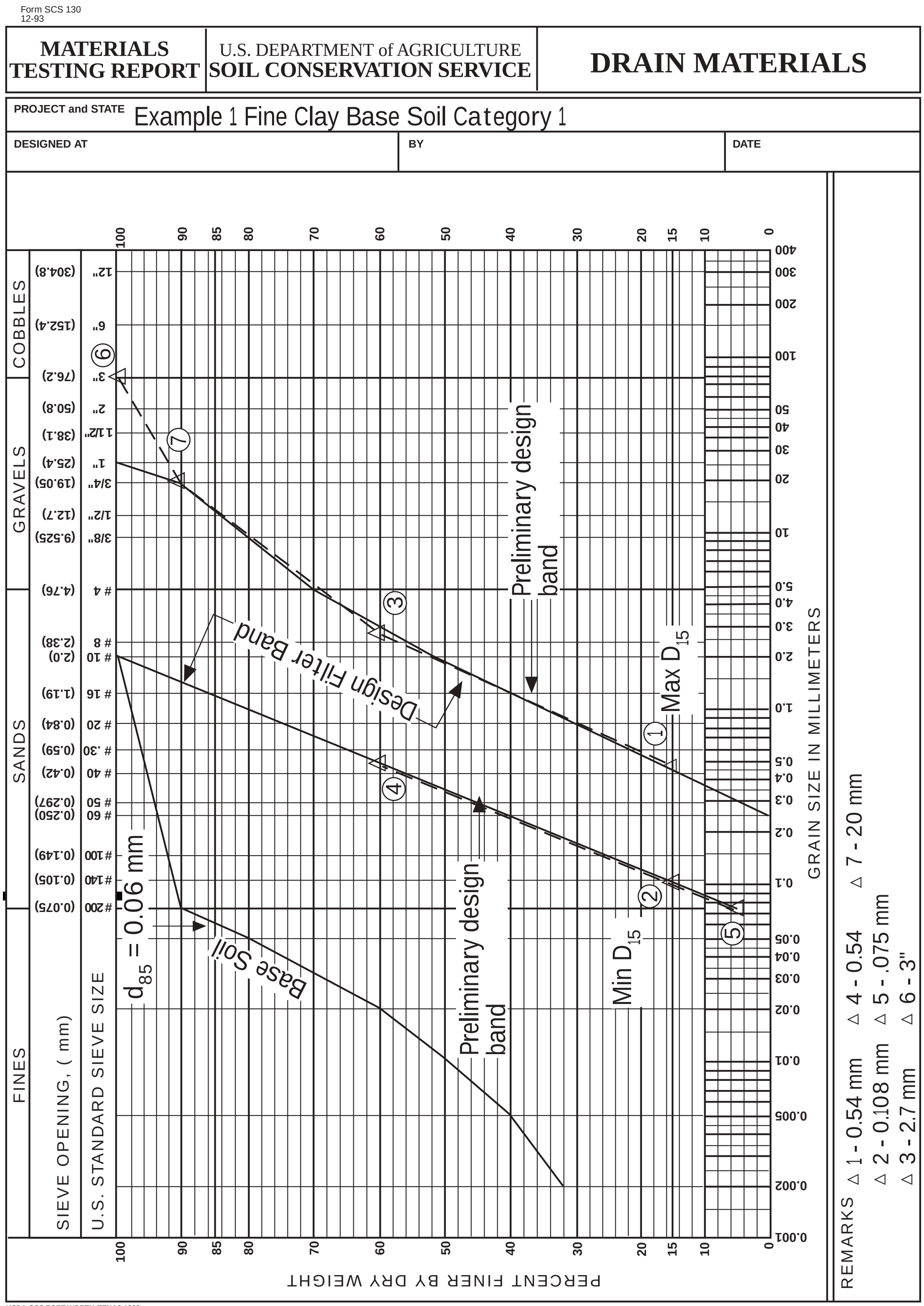
For this example, the filter will not be used around a perforated collector pipe, so step 12 is not applicable.

Additional design considerations: For this example, ASTM C-33 concrete sand falls well within the design band. Because this is a fairly standard, readily available gradation, no adjustments in the design band appear warranted. Selected ASTM Aggregate Specifications are given in appendix 26B.

Table 26–8 Design specification gradation for example 26–1 soil

Sieve size	% passing
1 inch	100
3/4 inch	90–100
No. 4	70–100
No. 10	52–100
No. 20	30–75
No. 60	0–40
No. 140	0–15
No. 200	0–5

Figure 26–1 Grain size distribution curve for fine clay base soil



Example 26–2 Silty sand with gravel base soil—Category 3

Given: The most important function of the filter being designed in this example is to act as a drain.

Step 1: Plot the gradation curve of the base soil material.

Refer to figure 26–2 for the plotted grain size distribution curve for this example silty sand with gravel base soil. The plotted curve is from the following data:

Sieve size	% passing
3 inch	100
1 inch	90
3/8 inch	82
No 4	78
No. 10	72
No. 20	66
No. 40	54
No. 100	32
No. 200	20
0.005 mm	4
0.002 mm	2

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

The example 26–2 base soil has particles larger than the No. 4 sieve, so the grain size distribution curve should be regraded on the No. 4 sieve. Proceed to step 3:

Step 3: Prepare adjusted gradation curves for base soils with particles larger than the No. 4 (4.75 mm) sieve.

Determine the regrading factor by dividing the value 100 by the percent passing the No. 4 (4.75 mm) sieve size. The regrading factor is:

$$\frac{100\%}{78\%} = 1.28$$

Using the original gradation analysis, plot a regraded curve for 100 percent passing the No. 4 (4.75 mm) sieve. The regraded percent passing values are equal to the original percent passing values times the regrading factor.

Sieve size	Original % passing	Regraded % passing
3 inch	100	—
1 inch	90	—
3/8 inch	82	—
No 4	78	100
No. 10	72	92
No. 20	66	85
No. 40	54	69
No. 100	32	41
No. 200	20	26
0.005 mm	4	5
0.002 mm	2	3

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table
26–1.

The example soil after regrading has 26 percent finer than the No. 200 sieve. From table 26–1, the soil is in category 3.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

The filtering criteria for base soil category 3 is (table 26–2): The maximum D_{15} of the filter will be less than or equal that given by the following expression:

$$D_{15} \le \left[\frac{(40 - A)}{(40 - 15)}\right]\left[(4)(d_{85}) - 0.7\text{ mm}\right] + 0.7\text{ mm}$$

Determine from the gradation curve of the regraded base soil that the d_{85} size is 0.84 mm. From the regraded curve, the value of A is 26 percent. Then the maximum D_{15} of the filter by the equation above is:

$$D_{15} \le \left[\frac{(40-26)}{(40-15)}\right]\left[(4)(0.84) - 0.7\text{ mm}\right] + 0.7\text{ mm}$$

$$\le 2.2\text{ mm}$$

This is labeled as Maximum D_{15} in figure 26–2.

Step 6: If permeability is a requirement (section 633.2603), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but not be less than 0.1 mm in any case.

The example 26–2 base soil has a d_{15} size of 0.032 before regrading. The minimum D_{15} of the filter is 4 x 0.032 = 0.128 (acceptable because it is larger than 0.1 mm). Label this value as Minimum D_{15} in figure 26–2.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in previous steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations:

Determine the ratio of the maximum D_{15} size to the minimum D_{15} sizes determined in previous steps. This ratio is:

$$\frac{2.2\text{ mm}}{0.13\text{ mm}} = 16.9$$

Because this ratio exceeds the criterion ratio of 5, adjustments are required in the values.

It was given that the most important function of the filter is to serve as a drain, so the maximum D_{15} is selected as the control point, equal to 2.2 mm. Label this value as Control point 1. To satisfy criteria, determine that the minimum D_{15} value is 1/5 of this value.

The minimum D_{15} value is then:

$$\frac{2.2\text{ mm}}{5} = 0.44\text{ mm}$$

Label this as Control point 2 in figure 26–2.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

The value for maximum D_{10} is calculated to be the maximum D_{15} size determine in step 7, divided by 1.2:

$$\frac{D_{15}}{1.2} = \frac{2.2}{1.2} = 1.83\text{ mm}$$

Calculate a value for the maximum D_{60}. The maximum D_{10} size times 6 is 1.83 x 6 = 11 mm. Label the maximum D_{60} size as Control point 3.

The minimum allowable D_{60} size is equal to the maximum D_{60} size divided by 5.

$$\frac{11}{5} = 2.2\text{ mm}$$

Label this as Control point 4 in figure 26–2.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26.5.

This table requires filters to have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–2.

It also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–2.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Determine that the minimum D_{10} size is equal to the minimum D_{15} size (determined in step 7) of 0.44 divided by 1.2:

$$\frac{0.44}{1.2} = 0.37 \text{ mm}$$

Because the value of minimum D_{10} size is less than 0.5 mm, the maximum D_{90} size is 20 mm (table 26–6). Label this value as Control point 7 in figure 26–4.

Step 11: Connect control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band.

Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

Refer to figure 26–2 for the completed filter band design. Table 26–9 gives the final design specified gradation. Note that all the control points are considered and that sieve sizes and corresponding percent finer values are selected to best fit the design band.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

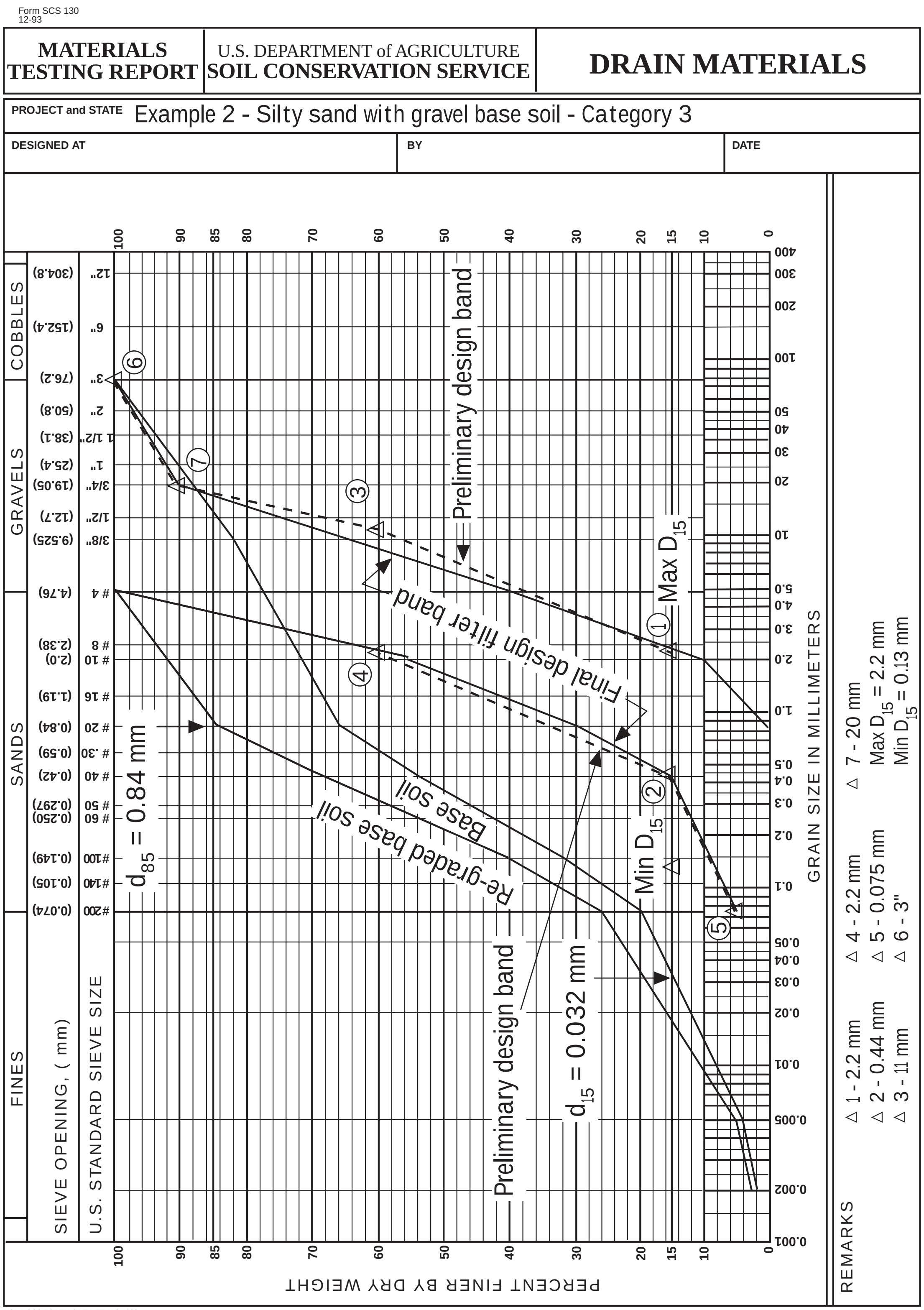
It is not given that this filter is to be used around a collector pipe, so this criterion is not applicable.

Additional design considerations: The design filter band does not coincide with standard, readily available aggregate gradations. Probably, a blend of standard aggregate gradations would be required to meet this design. Adjustments to the filter according to this step would not improve the availability. See following examples where this adjustment would be applicable. Using the design filter band, prepare the following tabular listing of the design.

Table 26–9 Design specification gradation for example 26–2 soil

Sieve size	% passing
3 inch	100
3/4 inch	90–100
1/2 inch	75–100
No. 4	40–100
No. 10	10–55
No. 20	0–30
No. 40	0–15
No. 100	0–9
No. 200	0–5

Figure 26–2 Grain size distribution curve for silty sand with gravel base soil—Category 3



Example 26–2A Silty sand with gravel base soil—Category 3

This example uses the same base soil as that in example 26–2. It is assumed that the most important function of the filter being designed is to act as a filter. Example 26–2 assumed the most important function was to act as a drain. Note the differences in the design steps.

Step 1: Plot the gradation curve of the base soil material. This step is the same as that in example 26–2. Refer to figure 26–2A for the plotted grain size distribution curve for this example silty sand with gravel base soil.

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve). Because the example 26–2 base soil has particles larger than the No. 4 sieve, the grain size distribution curve should be regraded on the No. 4 sieve. Proceed to step 3.

Step 3: Prepare adjusted gradation curves for base soils with particles larger than the No. 4 (4.75 mm) sieve. This step is the same as that for example 26–2. Refer to that example and see figure 26–2A.

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1. This step is the same as that for example 26–2. The soil is in category 3.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2. This step is the same as that for example 26–2. The maximum D_{15} size is 2.2 mm. This is labeled as Maximum D_{15} in figure 26–2A.

Step 6: If permeability is a requirement (section 633.2603), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The example 26–2A base soil has a d_{15} size of 0.032 mm before regrading. The value of minimum D_{15} of the filter is 4 x 0.032 = 0.128 mm (acceptable because it is larger than 0.1 mm). Label this value as Minimum D_{15} in figure 26–2A.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations.

Determine the ratio of the maximum D_{15} size to the minimum D_{15} sizes determined in previous steps:

$$\frac{2.2\text{ mm}}{0.13\text{ mm}} = 16.9$$

Because this ratio exceeds the criterion ratio of 5, adjustments are required in the values.

The most important function of the filter is to serve as a filter, so the minimum D_{15} is selected as the control point, equal to 0.13 mm. Label this Control point 2. To satisfy criteria, determine that the maximum D_{15} value is 5 times this value. The maximum D_{15} value is:

$$0.13 \text{ x } 5 = 0.65 \text{ mm}$$

Label this as Control point 1 in figure 26–2A.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

A value for maximum D_{10} is calculated by dividing the maximum D_{15} size (determine in step 7) by 1.2.

$$\frac{0.65}{1.2} = 0.54\text{ mm}$$

Calculate a value for the maximum D60 by multiplying the maximum D_{10} size times 6:

$$0.54 \text{ x } 6 = 3.24 \text{ mm}$$

Label the maximum D_{60} size as Control point 3.

The minimum allowable D_{60} size is equal to the maximum D_{60} size divided by 5:

$$\frac{3.24}{5} = 0.65 \text{ mm}$$

Label this as Control point 4 in figure 26–2A.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–2A.

It also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–2A.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

This table lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Calculate the minimum D_{10} size as equal to the minimum D_{15} size (determined in step 7) of 0.13 mm divided by 1.2:

$$\frac{0.13}{1.2} = 0.11 \text{ mm}$$

Because the value is less than 0.5 mm, the maximum D_{90} size is 20 mm (table 26–6). Label this value as Control point 7 in figure 26–2A.

Step 11: Connect control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band.

Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

Refer to figure 26–2A for the completed filter band design. The design is also tabulated in table 26–10.

Note that the control points are considered and that relatively even percent finer values are selected for standard sieve sizes for ease in writing specifications.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

It is not given that this filter is to be used around a collector pipe, so this criterion is not applicable.

Additional design considerations: The design filter band coincides fairly well with a standard, readily available aggregate gradation, ASTM C-33 fine aggregate for concrete. However, a slight adjustment in the filter design would make it more compatible with this standard gradation. The filter band can be adjusted to a more poorly graded configuration, a CU value of less than 6. Note that this is accomplished without violating other filtering or permeability criteria. Figure 26–2B shows how the original filter band design shown in figure 26–2A could be slightly altered to a steeper sloping band for the filter limits without violating any of the criteria previously covered.

The final filter design specification limits selected for example 26–2A, before and after possible adjustment, are shown in table 26–10.

Table 26–10 Design specification gradation for example 26–2A soil

Sieve size	Fig. 26–2A before adjustment (% passing)	Fig. 26–2B after adjustment (% passing)
3 inch	100	
3/4 inch	90–100	
1/2 inch	85–100	100
No. 4	70–100	80–100
No. 10	45–100	60–100
No. 20	20–65	20–100
No. 40	0–45	0–60
No. 60	0–30	0–35
No. 100	0–17	0–17
No. 200	0–5	0–5

Figure 26–2A Grain size distribution curve for silty sand with gravel base soil where primary function is filter

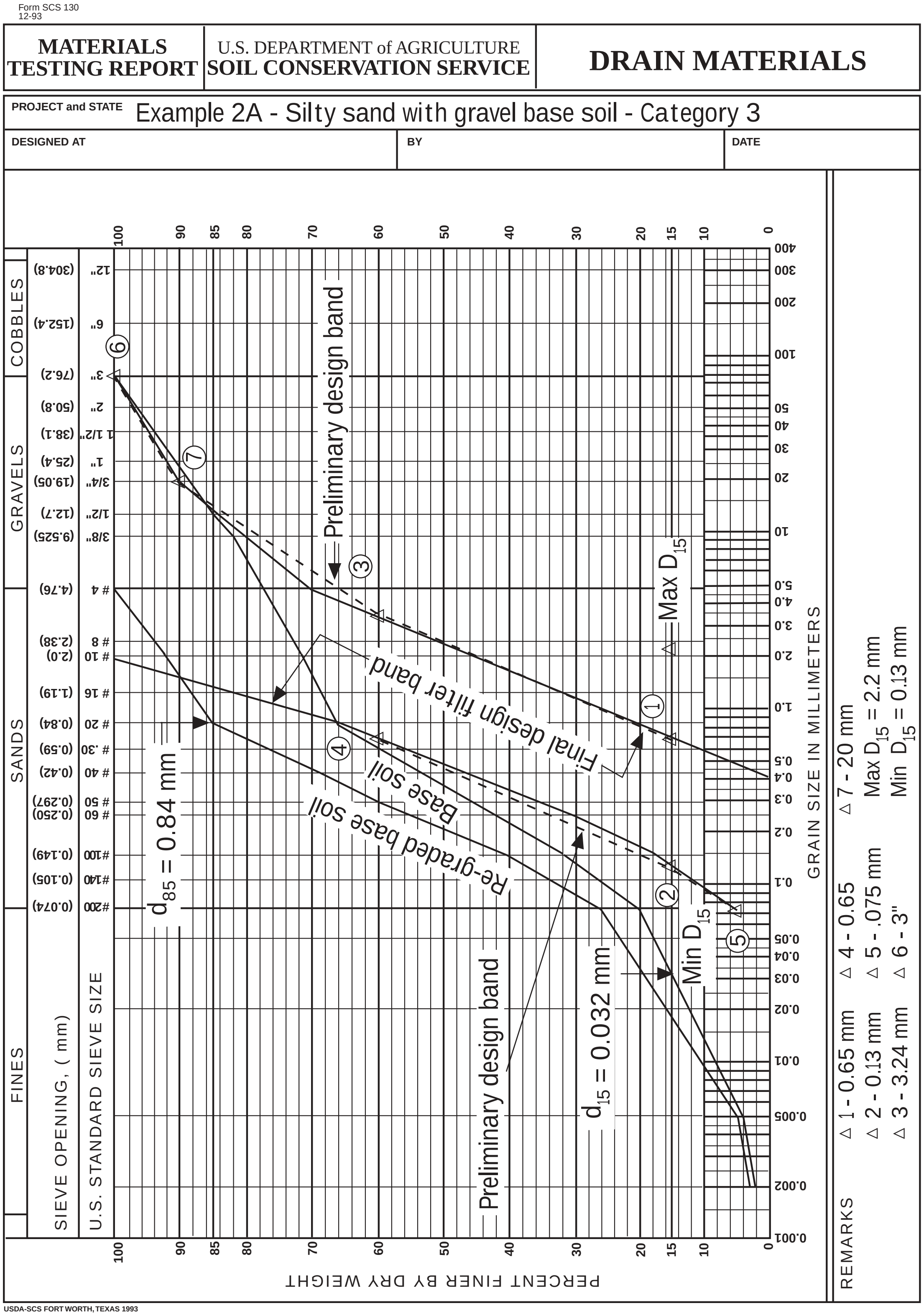
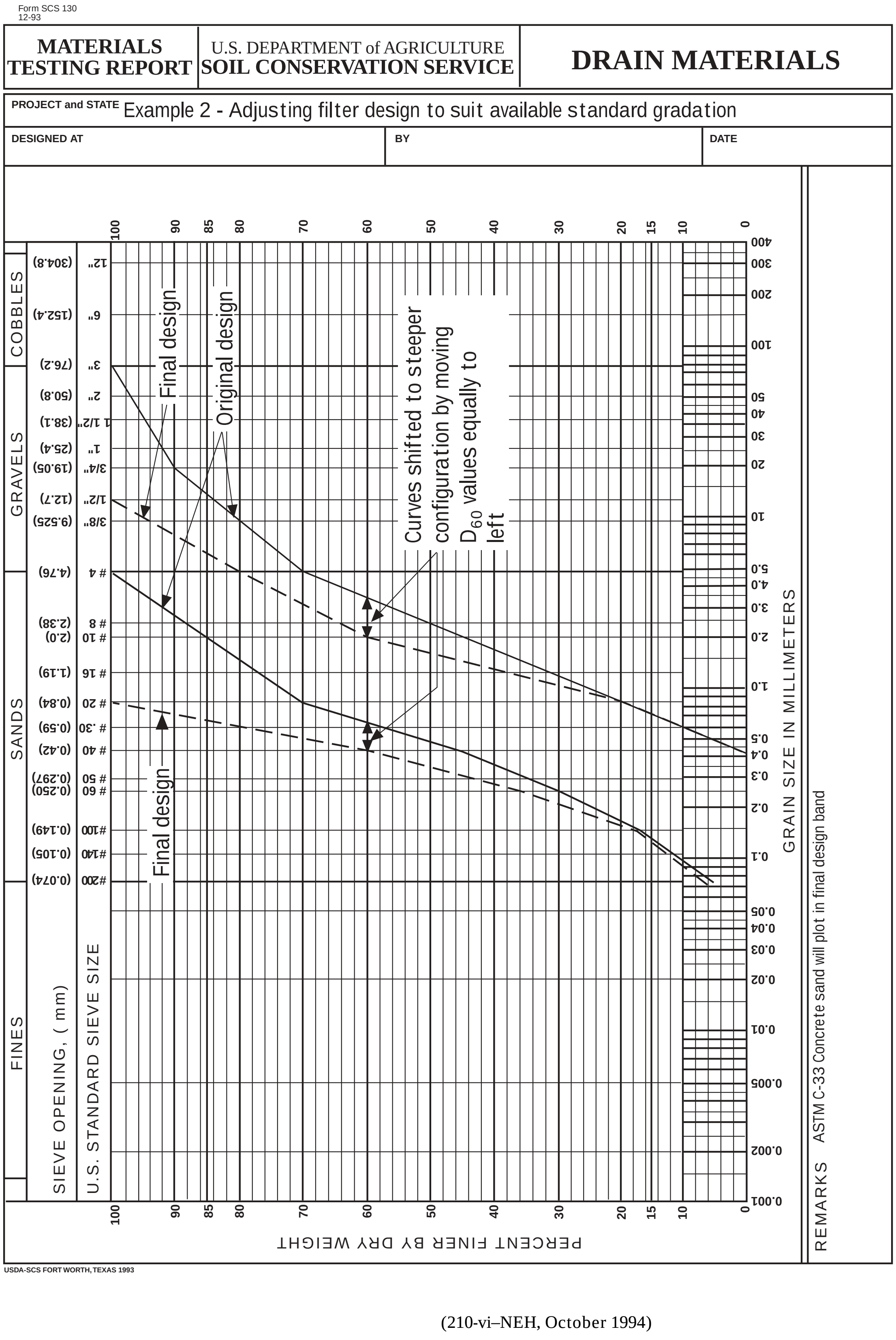


Figure 26–2B Grain size distribution curve for silty sand with gravel base soil (adjusting limits)



Example 26–3 Clayey gravel base soil—Category 2

Given: The most important function of the filter being designed is to act as a filter.

Step 1: Plot the gradation curve of the base soil material.

Refer to figure 26–3 for the plotted grain size distribution curve for this example clayey gravel base soil, labeled Base soil. The plotted curve is from the following data:

Sieve size	% passing
3 inch	100
1 inch	73
3/4 inch	66
1/2 inch	59
No. 4	47
No. 40	34
No. 60	31
No. 200	28
0.05 mm	26
0.02 mm	25
0.005 mm	18
0.002 mm	13

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

Because the example 26–3 base soil has particles larger than the No. 4 sieve, the grain size distribution curve should be regraded on the No. 4 sieve. Proceed to step 3.

Step 3: Prepare adjusted gradation curves for base soils with particles larger than the No. 4 (4.75 mm) sieve.

Determine the regrading factor by dividing the value 100 by the percent passing the No. 4 (4.75 mm) sieve size. The regrading factor is

$$\frac{100\%}{47\%} = 2.13$$

Using the original gradation analysis, plot a regraded curve for 100 percent passing the No. 4 (4.75 mm) sieve. The regraded percent passing values are equal to the original percent passing values times the regrading factor.

Sieve size	Original % passing	Regraded % passing
3 inch	100	—
1 inch	73	—
3/4 inch	66	—
1/2 inch	59	—
No. 4	47	100
No. 40	34	72
No. 60	31	66
No. 200	28	60
0.05 mm	26	55
0.02 mm	25	53
0.005 mm	18	38
0.002 mm	13	28

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

The example 26–3 base soil after regrading has 60 percent finer than the No. 200 sieve. From table 26–1, the soil is in category 2.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

This table shows the filtering criteria for base soil category 2 as follows. The maximum D_{15} of the filter will be less than or equal to 0.7 mm. This is labeled as Maximum D_{15} in figure 26–3.

Step 6: If permeability is a requirement (section 633.2602), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but will not be less than 0.1 mm in any case.

The example 26–3 base soil has a d_{15} size of about 0.0028 mm before regrading. Using the criterion, the minimum D_{15} of the filter would be 4 x 0.0028 = 0.011 mm. However, table 26–3 also shows that the minimum D_{15} is 0.1 mm. Label this value as minimum D_{15} in figure 26–3.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations:

Determine the ratio of the maximum D_{15} to the minimum D_{15} sizes:

$$\frac{0.7\text{ mm}}{0.1\text{ mm}} = 7$$

Because this value exceeds the criterion of 5, adjustment in the values is required. The most important function of this design filter is to act as a filter, so the minimum D_{15} value becomes controlling and is unchanged. Label this value Control point 2 in figure 26–3. Then, the maximum D_{15} value is 5 times this, or 5 x 0.1 mm = 0.5 mm. Label this as Control point 1 in figure 26–3.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less

Calculate a value for the maximum D_{10} size as equal to the maximum D_{15} size determined in Step 7 divided by 1.2:

$$\frac{0.5}{1.2} = 0.42\text{ mm}$$

The value for the maximum D_{60} is calculated using the maximum D_{10} size times 6:

$$0.42 \times 6 = 2.52\text{ mm}$$

Label the maximum D_{60} size as Control point 3.

The minimum allowable D_{60} size is then:

$$\frac{D_{60}}{5} = \frac{2.52}{5} = 0.50\text{ mm}$$

Label this as Control point 4 in figure 26–3.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–3.

Table 26–5 also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–3.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Table 26–6 lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Calculate a value for minimum D_{10} size by dividing the minimum D_{15} size determined in Step 7 by 1.2:

$$\frac{0.1}{1.2} = 0.083\text{ mm}$$

Because the value is less than 0.5 mm, the maximum D_{90} size is 20 mm (table 26–6). Label this value as Control point 7 in figure 26–3.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band. Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

See figure 26–3 for the final filter band design.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

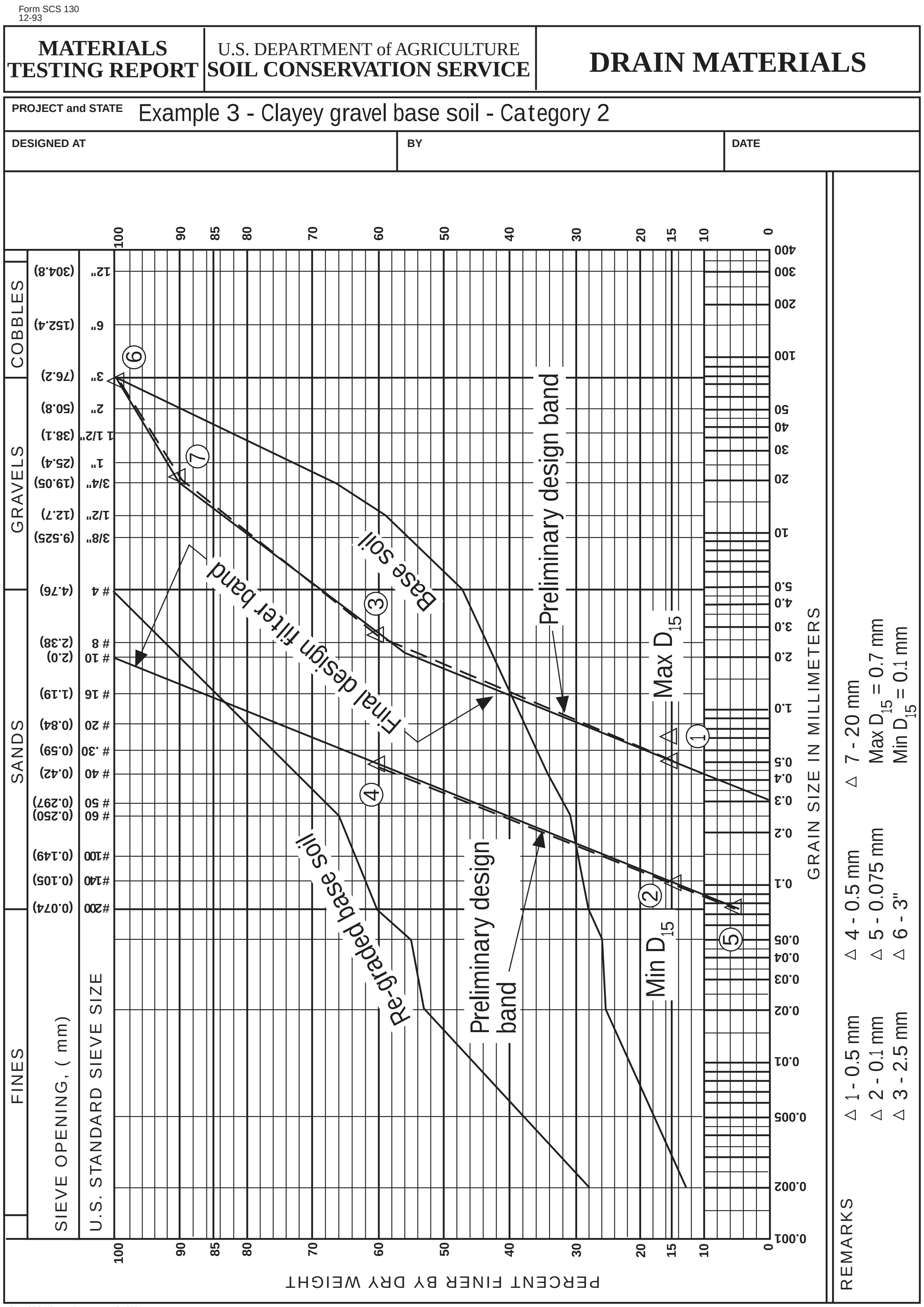
It is not given that this filter is to be used around a collector pipe, so this criterion is not applicable.

Additional design considerations: Standard Concrete Sand, ASTM C–33, plots within this final design band, so one may consider the design acceptable with no further modifications. If onsite sand or other cheaper filters could be located, some modification could be considered. Possible specification limits are shown in table 26–11.

Table 26–11 Design specification limits for clayey gravel base soil

Sieve size	% passing (1)
3 inch	100
3/4 inch	90–100
No. 4	70–100
No. 10	55–100
No. 20	30–75
No. 40	10–55
No. 50	0–45
No. 100	0–25
No. 200	0–5

Figure 26–3 Grain size distribution curve for clayey gravel base soil



Example 26–4 Silty sand base soil—Category 4

Given: The most important function of the filter being designed is to act as a filter.

Step 1: Plot the gradation curve of the base soil material.

Refer to figure 26–4 for the plotted grain size distribution curve for this example silty sand base soil, labeled Base soil. The plotted curve is from the following data.

Sieve size	% passing
No. 20	100
No. 40	94
No. 60	44
No. 140	14
0.05 mm	12
0.02 mm	10
0.005 mm	7
0.002 mm	4

Step 2: Proceed to Step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

Because the example 26–4 base soil has 100 percent of its particles finer than the No. 20 sieve, it has no particles larger than the No. 4 sieve. Therefore, the grain size distribution curve does not have to be regraded. Proceed to step 4.

Step 3: This step is not applicable because the base soil contains no particles larger than the No. 4 sieve. Go to step 4.

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

The example 26–4 base soil has 13 percent finer than the No. 200 sieve, determined from examination of the plotted grain size distribution curve in figure 26–4. From table 26–1, the soil is in category 4.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

The filtering criterion for base soil category 4 (table 26–2) is that the maximum D_{15} of the filter will be less than or equal to 4 times the d_{85} of the base soil.

The d_{85} of the base soil from the plotted grain size distribution curve in figure 26–4 is 0.39 mm. The maximum D_{15} is:

$$4 \times 0.39 \text{ mm} = 1.56 \text{ mm}$$

Label this as Maximum D_{15} in figure 26–4.

Step 6: If permeability is a requirement (section 633.2602), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but not be less than 0.1 mm in any case.

The example 26–4 base soil has a d_{15} size of 0.12 mm before regrading. Using the criterion, the minimum D_{15} of the filter would be 4 x 0.12 = 0.48. This is greater than the minimum required D_{15} of 0.1 mm, so it is acceptable. Label this value as Minimum D_{15} in figure 26–4.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations.

The ratio of the maximum D_{15} to the minimum D_{15} is:

$$\frac{1.56}{0.48} = 3.3$$

Because this value is less than the criterion value of 5, no adjustment is necessary. Label the maximum D_{15} and minimum D_{15} sizes as Control points 1 and 2, respectively, and proceed to the next consideration.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

Calculate a value for the maximum D_{10} size as equal to the maximum D_{15} size (determined in Step 7) divided by 1.2:

$$\frac{1.56}{1.2} = 1.3 \text{ mm}$$

Calculate a value for the maximum D_{60} by multiplying the maximum D_{10} size times 6:

$$1.3 \times 6 = 7.8 \text{ mm}$$

Label the maximum D_{60} size as Control point 3.

The minimum allowable D_{60} size is:

$$\frac{7.8}{5} = 1.56 \text{ mm}$$

Label this as Control point 4 in figure 26–4.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–4.

The table also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–4.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Table 26–6 lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Calculate a value for minimum D_{10} size by dividing the minimum D_{15} size determined in step 7 by 1.2:

$$\frac{0.48}{1.2} = 0.40 \text{ mm}$$

Because the D_{10} size is less than 0.5 mm, the maximum D_{90} size is 20 mm (table 26–6). Label this value as Control point 7 in figure 26–4.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a design for the coarse side of the filter band. Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

Refer to figure 26–4 for the selected filter band drawn. Table 26–12 lists the sieve/percent finer values selected.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

The filter is not being used adjacent to a collector pipe, so this step is not applicable.

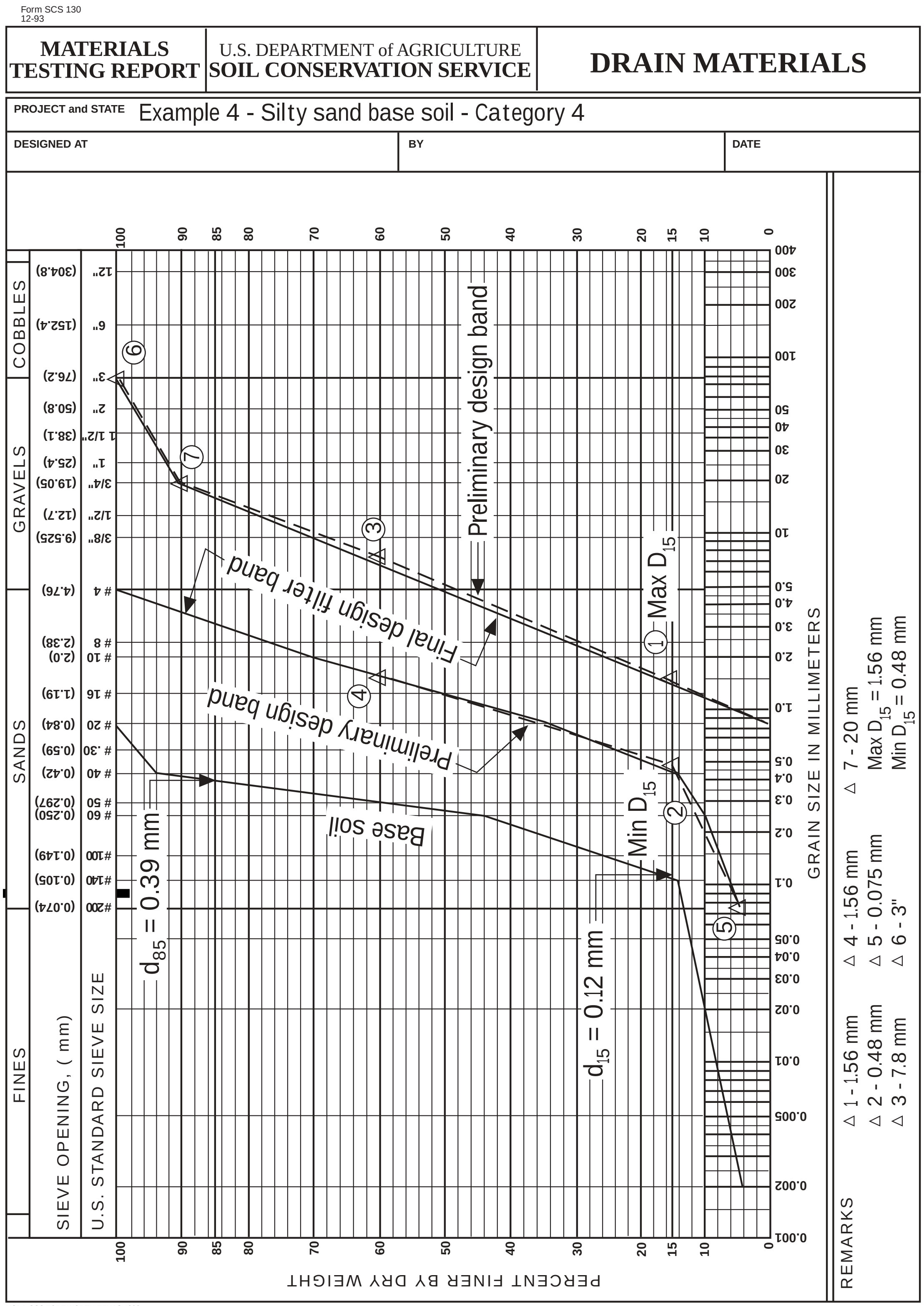
Additional design considerations: The specified filter band does not meet standard aggregate gradations. The band is more coarse than C–33 concrete sand, and it is finer than the standard gravel gradations (see appendix 26B). Possibly, the required filter gradation could be met by blending standard available gradations.

Consider adjustments in the steepness of the final design filter band shown in figure 26–4 if these adjustments would allow the use of such blends or other readily available gradations. The filter band may be adjusted to a steeper configuration, with a coefficient of uniformity of less than 6, but all the other criteria must still be met. Example 26–2A illustrated such an adjustment in the design filter band.

Table 26–12 The final selected design filter band gradation for silty sand base soil

Sieve size	% passing
3 inch	100
3/4 inch	90–100
No. 4	50–100
No. 10	25–70
No. 20	0–35
No. 40	0–14
No. 60	0–10
No. 200	0–5

Figure 26–4 Grain size distribution curve for silty sand base soil



Example 26–5 Design of a coarse filter to be compatible with a previously designed fine filter and used around a perforated pipe

The base soil for this example is the filter band obtained in the design for example 26–1. The base soil in this case is actually a band of soil gradations specifying a suitable sand filter. The sand filter was designed to protect a silty clay base soil.

Example 26–5 illustrates how to design a gravel filter band to be compatible with the finer sand filter previously designed. In the first part of this example it is understood that the gravel filter will not be used around perforated collector pipe, but some other type of outlet of seepage is employed. The second part of this example illustrates how the design of a coarse filter is changed if perforated pipe is used.

Step 1: Plot the gradation curve of the base soil material. In example 26–5, the base soil is actually a band of possible filter gradations. The filter band that was obtained in example 26–1 is used. Refer to the plotted grain size distribution curve for this example, labeled Fine filter in figure 26–5. The plotted band is from the following data:

Sieve size	% passing
1 inch	100
3/4 inch	90–100
No. 4	70–100
No. 10	52–100
No. 20	30–75
No. 60	0–40
No. 140	0–15
No. 200	0–5

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

Only the fine side of the specified filter band need be considered for this step because the finest base soil controls the filter criteria. Because the fine side of the filter band has no particles larger than the No. 4 sieve, step 3 is skipped. Proceed to step 4.

Step 3: Not applicable because the base soil contains no particles larger than the No. 4 Sieve.

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

Example 26–5 base filter band has from 0 to 5 percent finer than the No. 200 sieve, determined from examination of the plotted grain size distribution curve in figure 26–5. From table 26–1, the soil is in category 4.

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

This table states the filtering criteria for base soil category 4 as: The maximum D_{15} of the filter will be less than or equal 4 times the d_{85} of the base soil.

The finest gradation from the range of gradations given by the base filter band will be controlling under this criterion. The d_{85} of the fine side of the base filter band from the plotted grain size distribution curve in figure 26–5 is 1.2 mm. Then, 4 x 1.2 mm = 4.8 mm. This is labeled as Maximum D_{15} in figure 26–5.

Step 6: If permeability is a requirement (section 633.2602), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but not be less than 0.1 mm in any case.

The coarse limit of the base filter band will control under this criterion. Determine that the coarse limit line for the base filter band has a maximum d_{15} size of 0.45 mm. Using the criterion, the minimum D_{15} of the filter would be 4 x 0.45 = 1.8 mm. Label this value as Minimum D_{15} in figure 26–5.

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations.

The ratio of the maximum D_{15} to the minimum D_{15} is:

$$\frac{4.8}{1.8} = 2.7$$

Because this value is less than the criterion value of 5, no adjustment is necessary. Label the values of maximum D_{15} and minimum D_{15} as Control points 1 and 2, respectively, and proceed to step 8

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less.

Calculate a value for the maximum D_{10} size by dividing the maximum D_{15} size determined in Step 7 by 1.2:

$$\frac{4.8}{1.2} = 4.0 \text{ mm}$$

Calculate a value for the maximum D_{60} by multiplying the maximum D_{10} size times 6:

$$4.0 \times 6 = 24 \text{ mm}$$

Label the maximum D_{60} size as Control point 3.

To prevent an overly broad range of particle sizes in the filter, consider the requirement in step 7 that the ratio of maximum to minimum diameters be less than 5 for all percent passing values less than 60. The minimum allowable D_{60} size is:

$$\frac{24.0}{5} = 4.8 \text{ mm}$$

Label this as Control point 4 in figure 26–5.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–5.

Table 26–5 also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–5.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

This table lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Calculate the minimum D_{10} size by dividing the minimum D_{15} size determined in step 7 by 1.2:

$$\frac{1.8}{1.2} = 1.5$$

Because the D_{10} size is between 1.0 and 2.0 mm, the maximum D_{90} size is 30 mm (table 26–6). Label this value as Control point 7 in figure 26–5.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine limits of the filter band being designed. Connect Control points 6, 7, 3, and 1 to form the preliminary coarse limits of the filter band being designed. Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band, and tabulate the values.

Refer to figure 26–5 for the final coarse filter band designed for the condition of no perforated pipe being used. Note that the filter selected for final design has coefficient of uniformity values for the fine and coarse sides of the design bands slightly less than 6. The Control points 3 and 7 were shifted to the left slightly to have a smoother band shape. The data used for the designed filter band is given in table 26–13.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

For the second part of this example, it is assumed that the gravel filter being designed is for use around standard perforated drain pipe and is not for a critical drain. It is also given that rapid gradient reversal or surging is not predicted.

Standard perforations in drain pipe are 1/4 inch, plus-or-minus 1/16 inch. The maximum size of perforation that must be protected is then 5/16 inch, or about 8 mm. If the gravel filter being designed is to be used surrounding perforated pipe, an additional control point as defined by step 12 is necessary.

Design steps 1 through 11 are unchanged and not repeated here. The additional requirement of step 12 is that the D_{85} size of the filter may be no smaller than the perforation size for designs of noncritical drains where gradient reversal or surging is not predicted.

The additional design step 12 results in an additional control point labeled Control point 8. This is plotted in figure 26–5A. This additional control point is a minimum D_{85} size for the filter being designed and is equal to 8 mm, the maximum perforation size possible. Using Control point 8 does not significantly change the design for the coarse filter band.

Step 12 has different criteria if the coarse filter is designed for critical structure drains or for a situation where gradient reversal and surging were predicted with collector pipes. For this situation the coarse filter must have a D_{15} size no less the perforation size, 8 mm for the example. (For noncritical drains where surging is not predicted, the requirement is based on D_{85}.) In other words, this requirement is that the filter must be relatively coarse to prevent intrusion of the filter into the perforations in the high stresses present. However, filtering criteria require the gravel band to be a satisfactory filter for the sand filter (step 5) as well.

To accomplish this filtration function, the gravel must have a D_{15} of less than 4.8 mm. It is obvious then that one gravel filter cannot be used to satisfy both functions because both the criteria cannot be met. Another coarser filter that has a D_{15} greater than 8 mm must be designed to surround the perforations in the pipe and at the same time filter the gravel filter just designed. This is an example of the need for a 3-stage filter that could arise in critical flow situations.

Additional design considerations: Examine the limits of the gravel filter band constructed in figure 26–5. Note that the band is somewhat narrow at the lower percent passing sizes. Some designers have used an extended coarse filter limit as part of the specifications of the coarse filter band design to make it easier to supply the required filters (figure 26–5A).

The extended upper limits for a coarse filter are acceptable contingent upon the fine filter material actually used or delivered to a construction site, from the range of possible fine filters specified in the band being protected.

A gravel filter with a D_{15} size larger than the design filter band is acceptable if the fine sand filter actually delivered to a site has a d_{85} size larger than the minimum size possible within the design band of the fine sand filter. The coarse gravel filter actually used on the site must have a D_{15} less than or equal to 4 times the d_{85} size of the fine filter actually supplied from within the design band, based on the criteria in table 26–2 for Category 4 soils.

An extended coarse filter limit in the design band is used to provide maximum flexibility in obtaining filter materials. Where possible, specifications should fit readily available gradations from concrete aggregate suppliers to reduce cost of obtaining specially manufactured filter materials. However, criteria should not be relaxed because filter zones are important to the safe functioning of many structures.

Table 26–13 Data for designed filter band

Sieve size	% passing
3 inch	100
1 inch	90–100
1/2 inch	45–100
No. 4	15–60
No. 10	0–15
No. 20	0–10
No. 200	< 5

Figure 26–5 Gravel filter band design

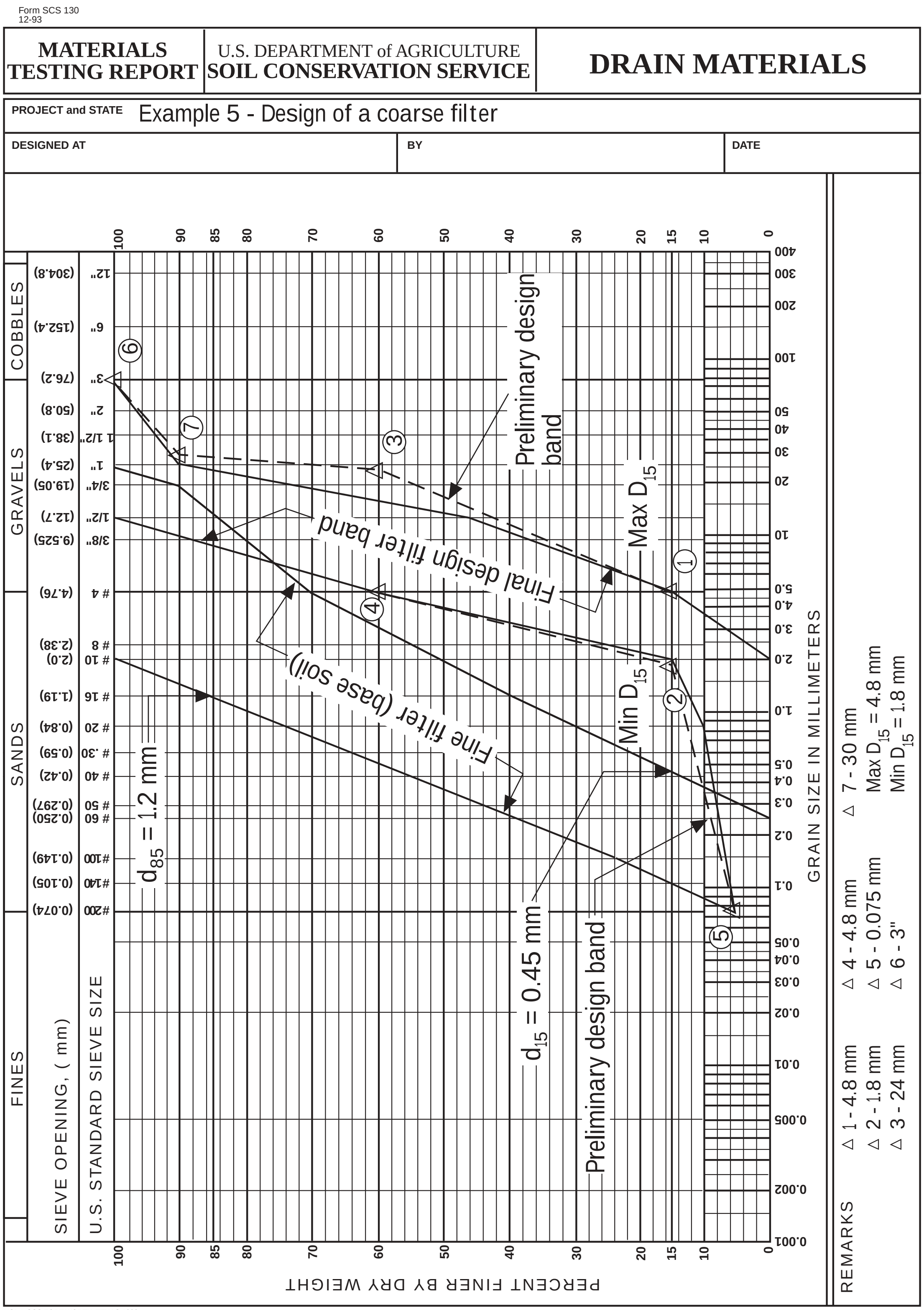
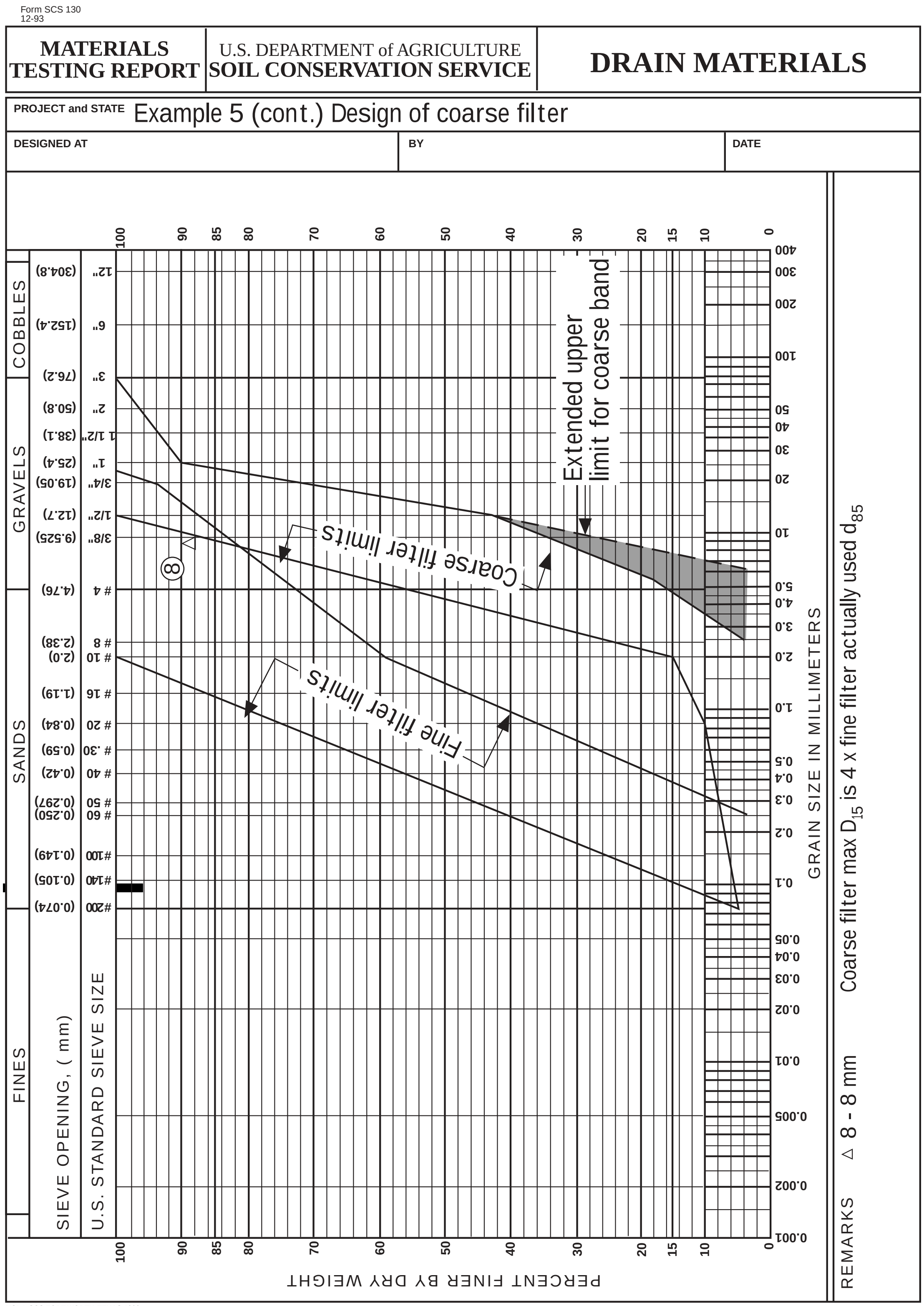


Figure 26–5A Gravel filter band design using an extended coarse filter limit



Example 26–6 Very fine clay base soil—Category 1

Given: The most important function of the filter being designed is to act as a filter.

Step 1: Plot the gradation curve of the base soil material.

Refer to figure 26–6 for the plotted grain size distribution curve for this example clay base soil, labeled Base soil. The plotted curve is from the following data:

Sieve size	% passing
No 4	100
No. 200	96
0.02 mm	90
0.005 mm	60
0.002 mm	34

Step 2: Proceed to step 4 if the base soil contains no gravel (material larger than the No. 4 sieve).

The example 26–6 base soil has 100 percent finer than the No. 4 sieve, and the grain size distribution curve does not have to be regraded. Proceed to step 4.

Step 3: Not applicable because the base soil contains no particles larger than the No. 4 sieve

Step 4: Place the base soil in a category determined by the percent passing the No. 200 (0.075 mm) sieve from the regraded gradation curve data according to table 26–1.

The example 26–6 base soil has 96 percent finer than the No. 200 sieve. The soil is in category 1 (table 26–1).

Step 5: To satisfy filtration requirements, determine the maximum allowable D_{15} size for the filter according to table 26–2.

This table shows the filtering criteria for base soil category 1 as: The maximum D_{15} of the filter will be less than or equal to 9 times the d_{85} of the base soil, but not less than 0.2 mm.

The d_{85} size of the base soil is 0.016 mm. Then, the maximum D_{15} of the filter will be less than or equal to 9 x 0.016 = 0.14 mm, but not less than 0.2 mm. Therefore, the maximum D_{15} of the filter is 0.2 mm. This is labeled Maximum D_{15} in figure 26–6.

Step 6: If permeability is a requirement (section 633.2602), determine the minimum allowable D_{15} according to table 26–3. Note: The permeability requirement is determined from the d_{15} size of the base soil gradation before regrading.

The permeability criterion for all categories of base soils is that the filter have a minimum D_{15} of no less than 4 times the d_{15} of the base soil (before any regrading of the base soil), but not be less than 0.1 mm in any case.

The example 26–6 base soil does not have a meaningful d_{15} size. The data shows that the base soil has 34 percent finer than 0.002 mm, the smallest commonly determined particle size. Therefore, use the default value of 0.1 mm for the minimum D_{15} of the filter. Label this value Minimum D_{15} in figure 26–6.

Step 7: The allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in steps 5 and 6 so that the ratio is 5 or less at any given percent passing of 60 or less. Adjustments may be required based on the following considerations.

For example 26–6, the ratio of the maximum D_{15} to the minimum D_{15} sizes is:

$$\frac{0.2}{0.1} = 2$$

Because the value is less than 5, no adjustment is needed in this step. The sizes selected become the maximum D_{15} and minimum D_{15} sizes for the final design filter band. These are labeled Control points 1 and 2, respectively, in figure 26–6. Go to step 8.

Step 8: The designed filter band must not have an extremely broad range of particle sizes to prevent the use of possibly gap-graded filters. Adjust the limits of the design filter band so that the coarse and fine sides of the filter band have a coefficient of uniformity of 6 or less. The width of the filter band should be such that the ratio of maximum to minimum diameters is less than or equal to 5 for all percent passing values of 60 or less

For example 26–6, calculate a value for maximum D_{10} by dividing the maximum D_{15} size of 0.2 mm determined in step 5 by 1.2:

$$\frac{0.2}{1.2} = 0.17 \text{ mm}$$

Calculate a value for the maximum allowable D_{60} size by multiplying the maximum D_{10} size by 6:

$$6 \times 0.17 = 1.02 \text{ mm}$$

Label this value as Control point 3 in figure 26–6.

Determine the minimum allowable D_{60} size for the fine side of the band by dividing the determined maximum D_{60} size by 5:

$$\frac{1.02}{5} = 0.20 \text{ mm}$$

Label this Control point 4 in figure 26–6.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to table 26–5.

This table shows that filters must have a D_5 greater than or equal to 0.075 mm, equal to the No. 200 sieve size. Label this value as Control point 5 in figure 26–6.

Table 26–5 also shows that filters must have a D_{100} of less than or equal to 3 inches. Label this value as Control point 6 in figure 26–6.

Step 10: To minimize segregation during construction, the relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using table 26–6. Label this as Control point 7.

Calculate the minimum D_{10} size of the preliminary filter band as equal to the minimum D_{15} value of 0.1 mm (obtained in step 6) divided by 1.2:

$$\frac{0.1}{1.2} = 0.083 \text{ mm}$$

Table 26–6 lists maximum D_{90} sizes for filters for a range of D_{10} sizes. Because the D_{10} value is less than 0.5 mm, the maximum D_{90} size is 20 mm (table 26–6). Label this value as Control point 7 in figure 26–6.

Step 11: Connect Control points 4, 2, and 5 to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 to form a partial design for the coarse side of the filter band. Complete the design of the filter band by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band, and tabulate the values.

Refer to figure 26–6 for an illustration of the complete filter design. Note that adjustments have been made in straight line portions of the design band to intercept even values for percent passing at standard sieve sizes. See the selected specified gradation in table 26–14.

Step 12: Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation size. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

Table 26–14 Design filter band data for example 26–6 soil

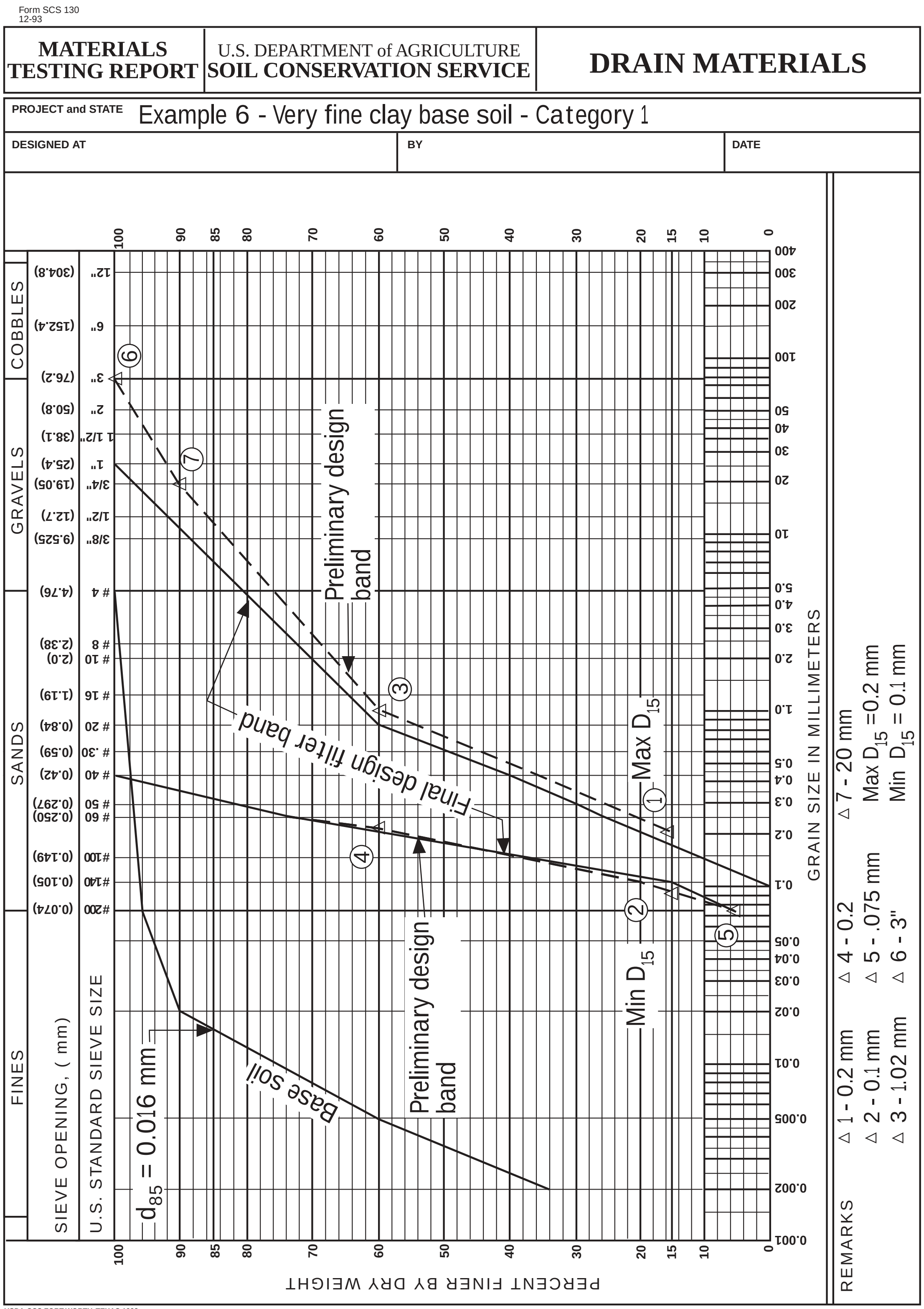
Sieve size	% passing
1 inch	100
No. 4	80–100
No. 10	70–100
No. 20	60–100
No. 40	40–100
No. 60	25–75
No. 140	0–15
No. 200	0–5

This step is then not applicable for this example because the filter will not be used around a perforated collector pipe. Table 26–14 lists the design filter band data obtained from the steps of this example.

Additional design considerations: ASTM C–33 fine concrete aggregate often meets the filter gradation requirements for many silts and clays. The base soil in example 26–6, however, is an unusual case in which the base soil is so fine that a filter finer than C–33 fine aggregate is required. Several alternatives are suggested for such situations:

- If a base soil having a d_{85} of 0.05 mm or larger is available at the site, using this soil in a core zone or in a transition zone between the core zone and the filter zone may be more economical. A more coarse filter could then be designed for the new base soil with the larger d_{85} size, and it is more likely that the specified gradation could be met with standard supplier sources.
- Attempt to locate a standard gradation that may fit the specified filter band. An example of such a gradation that might be located is ASTM D1073, Bituminous Mixture, Gradation No. 3. ASTM D1073 specifications for selected gradations are shown in appendix 26B.

Figure 26–6 Grain size distribution curve for very fine clay base soil



633.2604 Definitions

Base soil—The soil immediately adjacent to a filter or drainage zone through which water may pass. This movement of water may have a potential for moving particles from the base soil into or through the filter or drain materials.

d_{15}, d_{85}, and d_{100} sizes—Particle sizes (mm) corresponding respectively to 15, 85, and 100 percent finer by dry weight from the gradation curve of the base soil.

D_5, D_{10}, D_{15}, D_{30}, D_{60}, D_{85}, D_{90}, and D_{100} sizes—Particle sizes (mm) corresponding to the 5, 10, 15, 30, 60, 85, 90, and 100 percent finer by dry weight from the gradation curve of the filter.

Gradation curve (grain-size distribution)—Plot of the distribution of particle sizes in a base soil or material used for filters or drains.

Drain—A designed pervious zone, layer, or other feature used to reduce seepage pressures and carry water.

Filter—Sand or sand and gravel having a gradation designed to prevent movement of soil particles from a base soil by flowing water. Guidance on design using geotextiles and other nonsoil filter materials is not included.

Fines—That portion of a soil finer than a No. 200 (0.075 mm) U.S. Standard sieve as explained in table 26–1.

Soil category—One of four types of base soil material based on the percentage finer than the No. 200 (0.075 mm) U.S. Standard sieve as explained in table 26–1.

633.2605 References

Al-Hussaini, M.M. 1977. Contribution to the engineering soil classification of cohesionless soils. Misc. Pap. S-77-21, U.S. Army Corps. of Eng., Vicksburg, MS, November 1977.

Sherard, J.L. and L.P. Dunnigan. 1985. Filters and leakage control in embankment dams. *In* R.L. Volpe and W.E. Kelly (ed.). Seepage and leakage from dams and impoundments. Geotechnical Engineering Division Symposium Proceedings, Denver, CO, May 5, 1985. Amer. Soc. Civil Eng., New York, NY, pp. 1-30.

Sherard, J.L., and L.P. Dunnigan. 1989. Critical filters for impervious soils. Amer. Soc. Civil Eng., Journal of Geotechnical Engineering, No. 115 (7), July 1989, pp. 927-947.

Sherard, J.L., L.P. Dunnigan, and J.R. Talbot. 1984. Basic properties of sand and gravel filters. Amer. Soc. Civil Eng., Journal of Geotechnical Engineering, No. 110 (6), June 1984, pp. 684-700.

Sherard, J.L., L.P. Dunnigan, and J.R. Talbot. 1984. Filters for silts and clays. Amer. Soc. Civil Eng., Journal of Geotechnical Engineering, No. 110 (6), June 1984, pp. 701-718.

Talbot, J.R., and D.C. Ralston. 1985. Earth dam seepage control, SCS experience. *In* R.L. Volpe and W.E. Kelly (ed.), Seepage and leakage from dams and impoundments. Geotechnical Engineering Division Symposium Proceedings, Denver, CO, May 5, 1985. Amer. Soc. Civil Eng., New York, NY, pp. 44-65.

Appendix 26A Steps in Filter Design

1. Plot the gradations of base soils for which a filter is being designed on Form SCS-130 or acceptable alternative.

2. Determine the finest base soil that will control filter requirements. Also determine the soil with the most coarse limits that will control permeability requirements for the filter.

3. If the finest base soil has particles larger than the No. 4 sieve, regrade the soil on the No. 4 sieve.

4. Determine within which base soil category the regraded sample falls.

5. Determine the maximum D_{15} size based on filter criterion in criteria tables for that base soil category using the finest soil of the category plotted.

6. Determine the minimum D_{15} size based on permeability criterion in criteria tables, considering the coarsest sample plotted.

7. Calculate the ratio of the maximum D_{15} to the minimum D_{15} sizes from steps 5 and 6. If the ratio is less than or equal to 5, label the points Control points 1 and 2, respectively, on Form SCS-130, and continue to step 8. If the ratio is greater than 5, determine whether filtering or drainage is the most important function of the filter being designed. If filtering is most important, go to step 7A. If permeability is the most important consideration, go to step 7B.

7A. Filtering controls—Label the minimum D_{15} size as control point 2. Multiply minimum D_{15} by 5. This is the maximum D_{15} size; plot on Form 130 and label as control point 1. Go to Step 8.

7B. Permeability controls design—Label the maximum D_{15} size as Control point 1. Divide the maximum D_{15} size by 5. This is the minimum D_{15} size; plot on Form 130 and label as Control point 2. Go to Step 8.

8. Calculate a value for the maximum D_{10} size by dividing the maximum D_{15} size (Control point 1) determined in step 7 by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.) Calculate a value for maximum D_{60} by multiplying the maximum D_{10} size by 6. Label this as Control point 3.

Determine the minimum allowable D_{60} size for the fine side of the band by dividing the determined maximum D_{60} size by 5. Label this Control point 4.

9. Plot the minimum D_5 (for all filters) as equal to 0.075 mm (the No. 200 sieve). Label as Control point 5 on Form 130. Plot the maximum D_{100} (for all filters) as equal to 3 inches. Label as Control point 6 on Form 130.

10. Calculate a value for the minimum D_{10} size by dividing the minimum D_{15} size (Control point 2) determined in step 7 by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.)

Based on the determined value of minimum D_{10} size, obtain from table 26–6 the maximum allowable D_{90} size for the filter. Plot this value on Form 130 and label it as Control point 7.

11. Connect Control points 6, 7, 3, and 1 to form the coarse side of the initial filter design band. Connect Control points 4, 2, and 5 to form the fine side of the initial filter design band. Extrapolate the previously drawn lines to complete the preliminary fine and coarse limits of the preliminary filter band to 0 and 100 percent passing values. Adjust these limits to intercept relatively even values of percent passing at standard sieve sizes to simplify specifications (generally rounded at the nearest 5 on the percent passing scale) staying within the preliminary band. In most cases avoid sharp breaks in the design envelopes that might allow too broadly graded filter materials to be used in this final design step. If necessary to meet available gradations, adjust Control points 3 and 4 to the left, maintaining the ratio of diameters at 5, then draw other preliminary fine and coarse limits.

12. Design filters surrounding perforated pipe with an additional control point, determined as the minimum D_{85} size of the filter according to criteria tables. Label this value as Control point 8, and re-examine the design obtained in step 11.

A summary of the important criteria associated with the filter design process follows.

Base Soil Categories Summary

Base soil category	% finer than No. 200 sieve (0.075 mm) (After regrading, where applicable)	Base soil description
1	> 85	Fine silt, clays
2	40-85	Sands, silts, clays, silty and clayey sands
3	15-39	Silty and clayey sands, gravel
4	< 15	Sands, gravel

Filtering Criteria—Maximum D_{15}

Base soil category	Filtering criteria
1	$\leq 9 \times d_{85}$, but not less than 0.2 mm
2	≤ 0.7 mm
3	$\leq \left(\frac{40-A}{40-15}\right)\left[\left(4 \times d_{85}\right) - 0.7\text{mm}\right] + 0.7\text{mm}$ A = % passing No. 200 sieve after regrading (If 4 x d_{85} is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

Other Filter Design Criteria

To Prevent Gap-graded Filters

The width of the designed filter band should be such that the ratio of the maximum diameter to the minimum diameter, at any given percent passing value less than or equal to 60 percent, is less than or equal to 5. Both sides of the design filter band will have a coefficient of uniformity, defined as

$$CU = \frac{D_{60}}{D_{10}} \leq 6$$

Initial design filter bands by these steps have CU value of 6. For final design, filter bands may be adjusted so that CU values less than 6 result. This is acceptable as long as other filter and permeability criteria are satisfied.

Permeability Criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

Maximum and Minimum Particle Size Criteria

Base soil category	Maximum D_{100}	Minimum D_5 (mm)
All categories	< 3 inches (75 mm)	0.075 mm (No. 200 sieve)

(The minus No. 40 (.425 mm) material for all filters must be nonplastic as determined according to ASTM D4318.)

Segregation Criteria

Base soil category	If D_{10} is: (mm)	Then maximum D_{90} is: (mm)
All categories	< 0.5	20
	0.5–1.0	25
	1.0–2.0	30
	2.0–5.0	40
	5.0–10	50
	> 10	60

Criteria for Filters Used Adjacent to Perforated Collector Pipe

For noncritical drains where surging or gradient reversal is not anticipated, the filter D_{85} must be greater than or equal to the perforation size.

For critical drains, or where surging or gradient reversal is anticipated, the filter D_{15} must be greater than or equal to the perforation size.

Appendix 26B Standard ASTM Aggregate Specifications

Standard gradations for aggregates used in production of concrete are established by the American Society for Testing and Materials (ASTM). These aggregates are also commonly used for filter and drain zones in embankments, retaining walls, and other applications. Selected representative standard aggregates are listed in following tables for reference.

ASTM C-33—Standard Specification for Concrete Aggregates, lists standard gradations for both fine and coarse aggregates.

ASTM D-448—Standard Classification for Sizes of Aggregate for Road and Bridge Construction, lists standard gradations for only coarse aggregates.

ASTM D-1073—Lists standard gradations for Bituminous Mixtures.

In the interest of brevity, only selected representative standard gradations from the C–33 and D–1073 standards are listed in table 26B–1. A few gradations that may be useful are listed in D–448 and not in C– 33, but many of the gradations listed in the two standards are identical. Both of these ASTM standards are in Volume 04.02, Concrete and Aggregates.

Figure 26B–1 has plotted gradation bands for selected aggregates from the table.

Note: ASTM standards are periodically reviewed and updated, so use the latest version of the Standards for writing specifications. Refer to the latest ASTM standards volume to ensure that the gradations have not changed from those listed in table 26B–1 or to determine other standard gradations not listed. This table only lists selected representative gradations.

Table 26B–1 Selected standard aggregate gradations

Fine aggregate—ASTM C–33

ASTM	Percent finer than sieve no.							
size	#200	#100	#50	#30	#16	#8	#4	3/8"
Fine	3-5*	2–10	10–30	25–60	50–85	80–100	95–100	100

Coarse aggregates—ASTM C–33

ASTM	Percent finer than sieve no.								
size	#16	#8	#4	3/8"	1/2"	3/4"	1"	1-1/2"	2"	3"
357	—	—	0–5	—	10–30	—	35–70	—	95–100	100
56	—	—	0–5	0–15	10–40	40–85	90–100	100		
57	—	0–5	0–10	—	25–60	—	95–100	100		
67	—	0–5	0–10	20–55	—	90–100	100			
7	—	0-5	0-15	40-70	90-100	100				
8	0–5	0–10	10–30	85–100	100					

See the footnote at the end of the table.

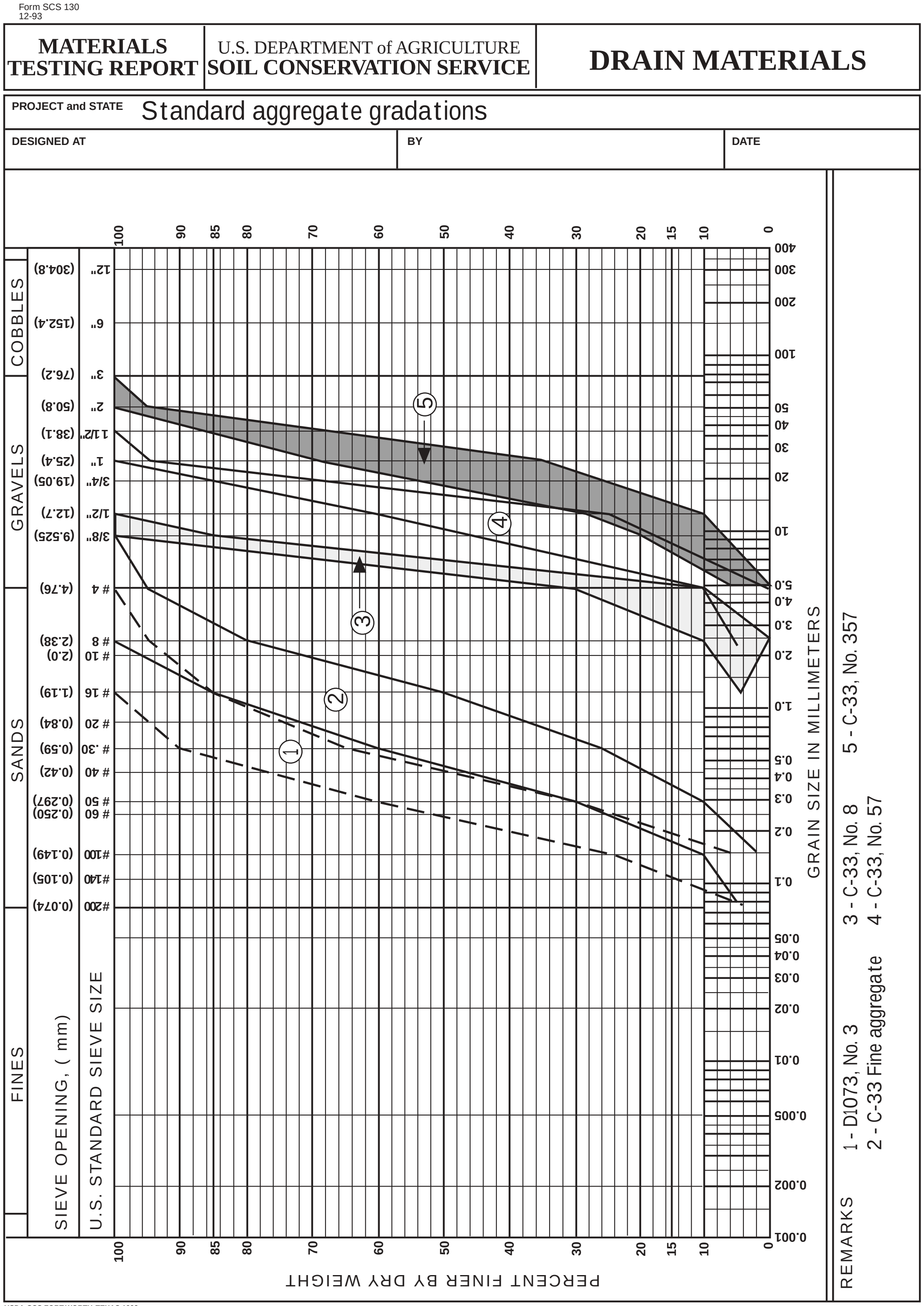
Table 26B–1 Selected standard aggregate gradations—Continued

Bituminous mixtures—ASTM D–1073

ASTM mix	Percent finer than sieve no. #200	#100	#50	#30	#16	#8	#4	3/8"
2	0–5	0–12	8–30	28–52	50–74	75–100	100	
3	0–5	5–25	30–60	65–90	85–100	95–100	100	
4	0–10	2–20	7–40	20–65	40–80	65–100	80-100	100

* For concrete aggregate, the permissible percentage finer than the No. 200 sieve is 3 to 5 percent, depending on the abrasion resistance desired for the manufactured concrete. In the case of manufactured sand, if the material finer than the No. 200 sieve consists of the dust of fracture, essentially free of clay or shale, these limits may be increased to 5 and 7 percent respectively. For drain and filter applications, the percentage finer than the No. 200 sieve is specified according to SM Note 1 as less than or equal to 5 percent, and an additional requirement is that the fines (minus No. 40 sieve) are nonplastic.

Figure 26B–1 Standard aggregate gradations



U. S. Department of Agriculture
Soil Conservation Service
Engineering Division
Soil Mechanics Unit

May 1971

Soil Mechanics Note No. 3: Soil Mechanics Considerations for Embankment Drains

I. Purpose and Scope

This Soil Mechanics Note is a guide for the design of drainage for embankments and associated foundations. Each drain type is related to applicable site conditions so that the appropriate type or types may be incorporated in a drainage system. Recommended processes are given for determining drain dimensions and outlet sizes. In the procedures presented, seepage quantities to be drained and permeability coefficients of materials involved are knowns. Examples are given in Appendix C.

II. Definitions

A. Interceptor drain - a drain that physically intercepts flow paths or fully penetrates water bearing strata.

B. Pressure relief drain - a drain that produces an area of low pressure to which water will flow from adjacent areas of higher pressure.

C. Filter material - a layer or combination of layers of pervious materials designed and installed in such a manner as to provide for water movement, yet prevent movement of soil particles due to flowing water.

D. Drain material - sand, gravel, or rock that has specific gradation limits designed for required permeability and internal stability.

E. Base material - any material (embankment, backfill, foundation or other filter layer) through which water moves into a drainage system.

F. Coefficient of permeability - the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions.

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Comments by M. M. Culp, Chief, Design Branch, and R. S. Decker, Head, Soil Mechanics Unit, were very helpful.

III. Functions of drains.

Drains are included in embankments and foundations for two basic reasons:

A. To prevent piping by controlling migration of soil particles under seepage flow. Materials fulfilling the requirements of Soil Mechanics Note No. 1 will control migration.

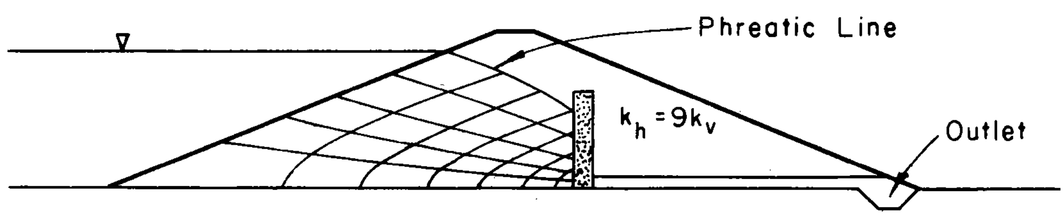
B. To control pressure build-up by providing adequate capacity to carry the seepage flow.

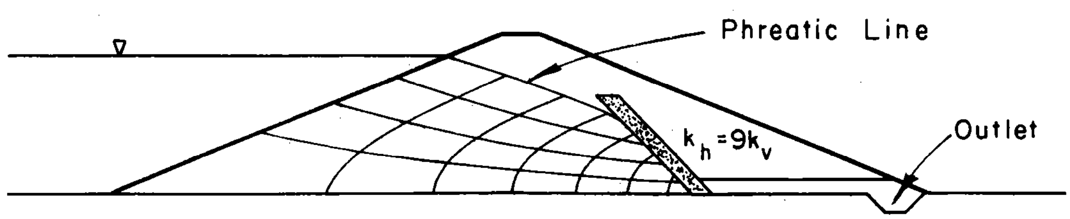
There are no hard and fast rules for selecting a reasonable margin of safety for drain design. Judgment in this respect must be related to (1) past experience with similar materials, (2) the detail used in site investigation and testing programs, and (3) the limitations that analyses have in representing site conditions.

Some individuals prefer to estimate seepage quantities as realistically as possible and factor these quantities for the design discharge. Others prefer to apply a factor to the drain dimensions as the final step. There are also many situations where ample capacity can be provided by selecting a highly pervious drain material. A factor of ten is often used. However, this should not be accepted across the board because there are situations where a lesser margin is adequate and there are situations where a greater margin is needed. Regardless of the approach used, the designer must be careful not to compound safety by entering a factor into each of the steps involved in the design process.

IV. Types of drains and their application.

A. Vertical and sloping embankment drains.





Vertical and sloping embankment drains are primarily interceptors that provide positive control of embankment seepage.

1. Site conditions where applicable.

   a. Embankment material not susceptible to cracking: In this case, water that percolates through the soil is intercepted to insure that seepage does not occur in materials downstream from the drain. This applies when:

      (1) The horizontal permeability of the embankment is significantly higher than the vertical permeability of the embankment. It is not possible to obtain isotrophy in embankments constructed from fine-grained soils or from coarse-grained soils that contain fines. This is due in part to construction methods but mostly to non-uniformity in soil deposits. The degree of anisotrophy to use in design is a matter of judgment because there is no good way to determine this property either before or after construction. The following table, which is from "Earth and Earth Rock Dams" by James L. Sherard et al, 1963, John Wiley and Sons, Inc., page 368, is considered to be a conservative guide.

Description of Soil in Borrow Area	k_h/k_v
Very uniform deposit of fine-grained soil (CL and ML)	9
Very uniform deposit of coarse soils with fines (GC and GM)	25
Very erratic soil deposits	100 or higher

      (2) Stability and/or durability of downstream embankment material is such that it cannot be allowed to saturate.

      Variability of soils in many borrow sources is so great that the engineering properties of the resulting fill cannot be determined with any reasonable degree of accuracy. It may be more economical to place these materials in a "random fill" zone downstream from a positive drain than to either waste them or disregard them altogether.

If used, materials suspect of undergoing marked and unpredictable changes upon saturation should be placed where they cannot saturate. Soils containing concentrations of soluble salts and some of the "degradable" shale derivatives are examples.

b. Embankment material susceptible to cracking: In this case, water which comes primarily through cracks formed within the embankment is intercepted to prevent piping and insure overall safety of the dam. This applies when:

(1) Cracks develop as a result of movements (differential settlement, seismic, etc.).

(2) Cracks develop as a result of desiccation.

(Note: Other factors may contribute to development of cracks.)

2. Information required from the investigation.

a. Index properties of base materials.

b. Information needed to evaluate settlement profiles.

(1) Boundaries of compressible foundation soils and of bedrock surfaces.

(2) Compressibility of embankment and foundation soils.

(3) Water table conditions and drainage characteristics of foundation soils.

c. Factors contributing to desiccation cracking such as climatic conditions and shrink-swell characteristics of embankment soils.

d. Earthquake potential.

e. Permeabilities of base materials.

f. Gradations and permeabilities of available drain materials.

3. Design procedures and considerations.

   a. Embankment material not susceptible to cracking. Use Figure No.. 1, which is based on flow net solutions, for proportioning the drain. If the slope is steeper than 1/2:1, use values for a slope of 1/2:1.

   b. Embankment material susceptible to cracking. Design depends on the following:

      (1) The drain must have sufficient thickness so it will not be disrupted by the amount of movement that can occur. A minimum horizontal thickness of 10 feet is suggested on 1:1 slopes and steeper. Horizontal thickness should be increased on flatter slopes.

      (2) Drain material must be internally stable and self-healing (well graded), with D_{85} size > 2 in. Care must be taken to prevent segregation.

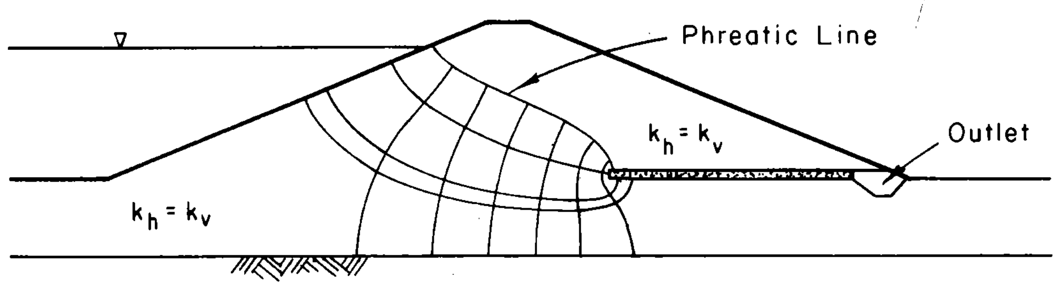
      (3) Drain material must be free flowing and deformable without cracking (clean and free of any cementing materials).

      (4) Drain materials must be pervious enough to remove anticipated flow in the cross-sectional area provided.

      (5) Drain material must be graded to control migration of base materials. When it is not possible to meet this requirement with a single drain material, an appropriate filter with a minimum horizontal thickness as suggested for the drain fill in (1) above will be provided in addition to the drain material.

   Note: An example is not included in Appendix "C". Special study is required when cracking is anticipated.

B. Horizontal blanket drain.



The horizontal blanket drain is primarily a pressure relief drain placed in the downstream area of an embankment.

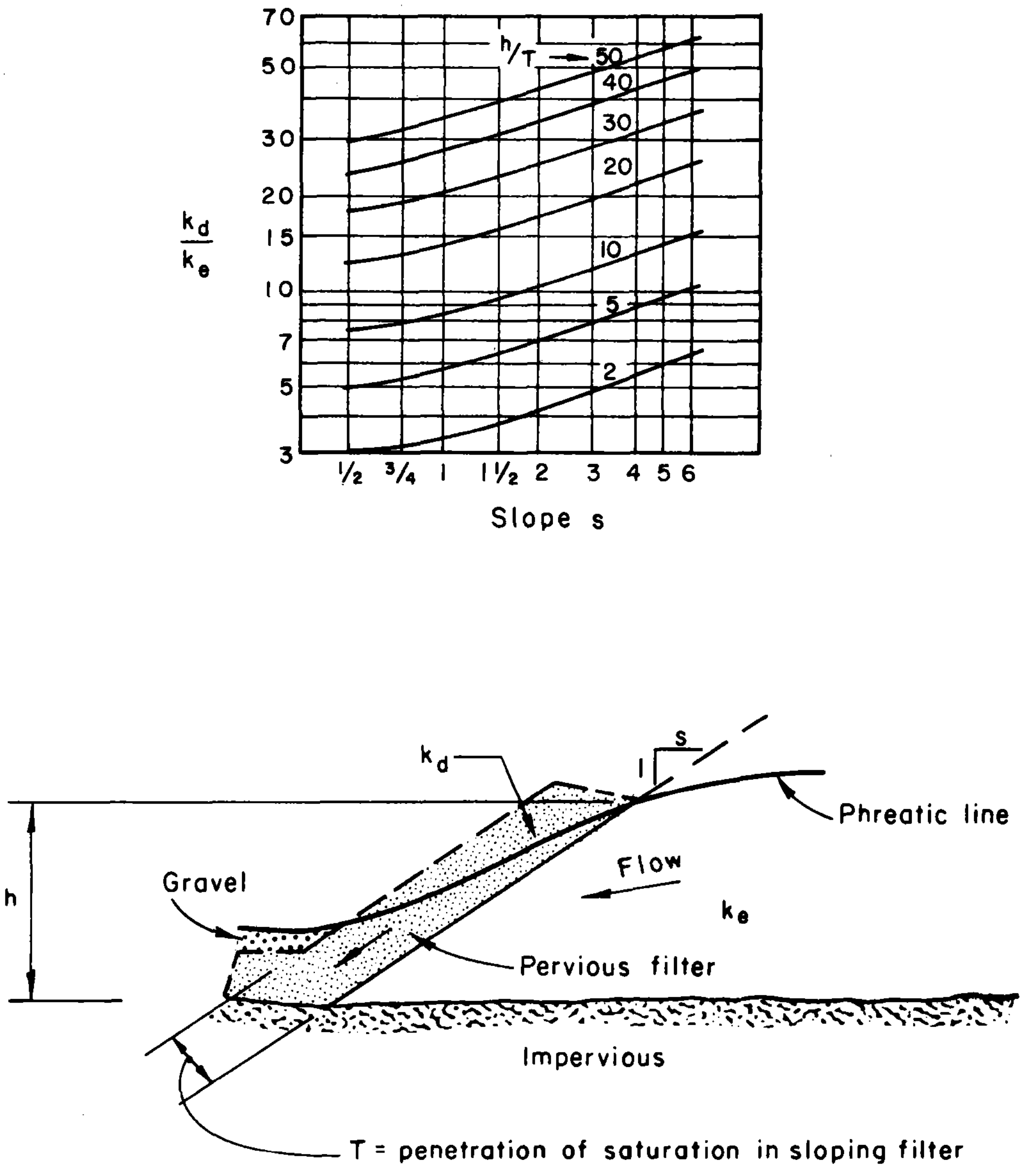


Figure 1. Flow net solution for seepage into sloping filters on various slopes. (Adapted from Harry R. Cedergren, Seepage, Drainage and Flow Nets, 1967, John Wiley and Sons, Inc., page 195, Fig. 5.10)

1. Site conditions where applicable.

   a. When there is no significant difference between the vertical and horizontal permeabilities of the embankment and/or the foundation.

   b. When bedrock is pervious (drain placed directly on bedrock).

   c. When a good bond cannot be obtained between impervious bedrock and the embankment.

2. Information required from the investigation.

   a. Extent and elevation of the water table.

   b. Index properties of the base materials.

   c. Extent and configuration of the base materials including the location of impervious boundaries.

   d. Permeabilities of base materials and condition of the bedrock.

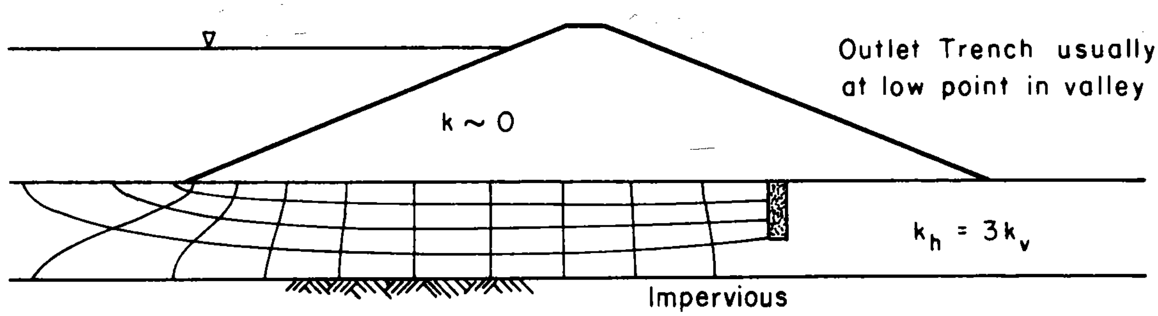
   e. Gradations and permeabilities of available drain materials.

3. Design procedures.

   Use Darcy's law, q = kiA, for solution.

4. Flow in blanket drains placed on abutments is essentially down slope. Information required from the investigation is the same as that required for horizontal blanket drains. Design procedures outlined for vertical embankment drains are applicable, i.e., Figure 1 or Darcy's law can be used.

C. Foundation trench drain.



The foundation trench drain is primarily a pressure relief drain. It is most effective when it penetrates all pervious strata.

1. Site conditions where applicable.

   a. When horizontal permeability of the foundation is significantly greater than vertical permeability of the foundation.

   b. To relieve pressure from foundation aquifers.

   c. To control pipable foundation materials.

2. Information required from the investigation.

   a. Extent and elevation of the water table.

   b. Magnitude of water pressure in any aquifers.

   c. Index properties of base materials.

   d. Thickness of base materials and their position.

   e. Continuity or discontinuity of base materials (upstream, downstream and across the valley) and the location of impervious boundaries.

   f. Permeabilities of the base materials.

   g. Gradations and permeabilities of available drain materials.

3. Design procedures.

   a. Foundation trench drains without pipe. Use Darcy's law, $q = kiA$.

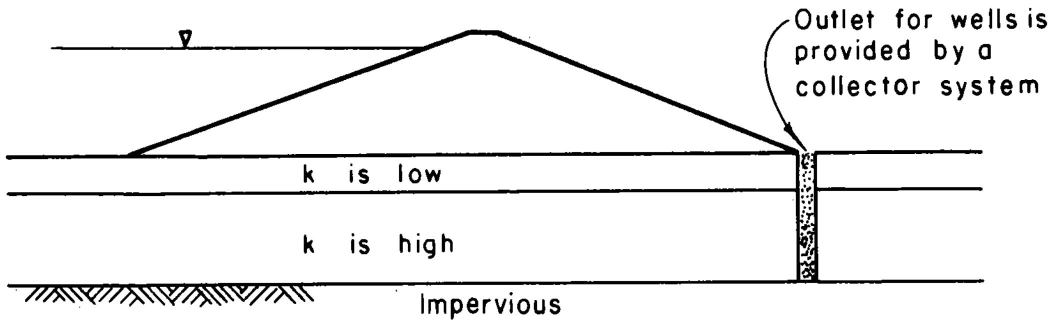
   b. Foundation trench drains with pipe.

      (1) Proportion the drain fill to carry at least 50% of the design discharge.

      (2) Proportion the pipe to carry at least 50% of the design discharge with the pipe 3/4 full.

   There is not much information in the literature on capacity of drains with pipes that applies to dams. Appendix "A" contains a brief review of a few studies that have some application and concludes with a suggested design approach for perforated pipe placed in gravel drain material.

D. Relief wells.



Relief wells are pressure relief drains. They are generally located near the downstream toe of an embankment for accessibility.

1. Site conditions where applicable.

   Relief wells are particularly adapted for control of pressures from confined aquifers that are too deep to drain with trenches including deep, stratified alluvial deposits having significant differences in permeability of the various strata.

2. Information required from the investigation.

   a. Extent and elevation of the water table.

   b. Magnitude of water pressures within the aquifers.

   c. Index properties of base materials.

   d. Thickness of the aquifer and the confining materials.

   e. Continuity or discontinuity of the aquifer and the confining materials (upstream, downstream, and across the valley), including the location of impervious boundaries.

   f. Permeability of the aquifer and the confining materials.

   g. Gradations and permeabilities of available sand or gravel pack materials.

3. Design procedures.

   a. Deferred action approach.

      When it is either impractical or impossible to evaluate all the factors in Section 2 above to the degree necessary for design of relief wells during the design stage, proceed as follows:

(1) Install piezometers during construction so that pressure relationships may be established for the critical areas.

(2) Monitor pressures until they stabilize under a given reservoir level (a level that is believed to be safe).

(3) Compare measured pressures to allowable pressures and evaluate need for relief (measured pressures may have to be adjusted to full reservoir head).

(4) When needed, design the relief well system using measured or adjusted pressures and the procedures given in "Design of Finite Relief Well Systems", Corps of Engineers EM 1110-2-1905 dated March 1, 1963, or the procedures outlined in Appendix B.

b. Design prior to construction.

When all of the factors in Section 2 above can be evaluated reasonably well or conservatively estimated prior to design:

(1) Estimate uplift in critical areas (usually along the downstream toe). Methods similar to those given in "The Effect of Blankets on Seepage Through Pervious Foundations", by P. T. Bennett, ASCE Transactions, Vol. 111, 1946, and in the SM-10 Manual, Chapter 12, pages 12-19 to 12-21, may be used to estimate uplift pressures. These methods should not be used when there is insufficient evidence from the investigation to prove that an aquifer is continuous for considerable distances upstream and downstream from the dam. When it is known that continuity does not exist, the only recourse is to estimate uplift pressures conservatively.

(2) If uplift is detrimental, base the design on procedures given in "Design of Finite Relief Well Systems", Corps of Engineers EM 1110-2-1905 dated March 1, 1963, or those given in Appendix B.

Note: Design changes may be needed when additional information becomes available during construction or after the structure is in operation, even though all factors appeared to be clear-cut at the time of design.

V. Drain Outlets

A drain outlet is a section of the system that has the primary purpose of conducting accumulated seepage to a controlled discharge point.

A. Types

1. Transverse (essentially perpendicular to the embankment centerline)

a. Outlet for foundation trench drain.

b. Outlet for vertical embankment drain.

c. Outlet for abutment drains.

d. Outlet for springs.

2. Longitudinal (essentially parallel to the embankment centerline)

a. Outlet for a blanket drain (usually placed at the downstream toe).

b. Outlet for relief wells.

B. Design procedures for outlets are similar to those presented for drains.

VI. Special Situations

A. Embankment zones. When an embankment zone is to function as a drain, material placed in that zone must meet the permeability and piping requirements for drain material. On-site materials generally contain enough fines to limit permeability. Permeability determinations and flow nets will provide guidance on the effectiveness of these materials for drainage zones.

B. Springs. It may be necessary to increase the capacity of drains to accommodate flow from springs. In many cases, it is desirable to provide separate drainage outlets for springs.

C. External abutment drains. Drains outside the limits of an embankment will be designed by the procedures outlined for drains placed under embankments.

D. Abutment well drains. These are either horizontal or slanted wells for drainage of deeply fractured rock abutments and other deep, pervious abutment materials. Design procedures are outside the scope of this note.

E. Compressible foundations. When drains with pipes are placed on or in compressible foundation soils, settlement profiles will be evaluated and pipe grades adjusted to accommodate for settlements.

Appendix A

Pipes in Drains

This appendix contains an approach for sizing of pipes installed in drains. Two flow conditions are considered: (1) flow through openings into the pipe is based on orifice flow with an area reduction to account for blockage by particles and (2) flow in the pipe is based on open channel flow.

Review of many papers dealing with pipes in drains yielded only a small amount of data on head-discharge relationships for perforations or slots. Information from three studies is plotted on Figure A-1, Head-discharge relationship for pipe perforations. Gradation of the drain material surrounding these pipes is shown on Figure A-2. Comments on the head-discharge curves are as follows:

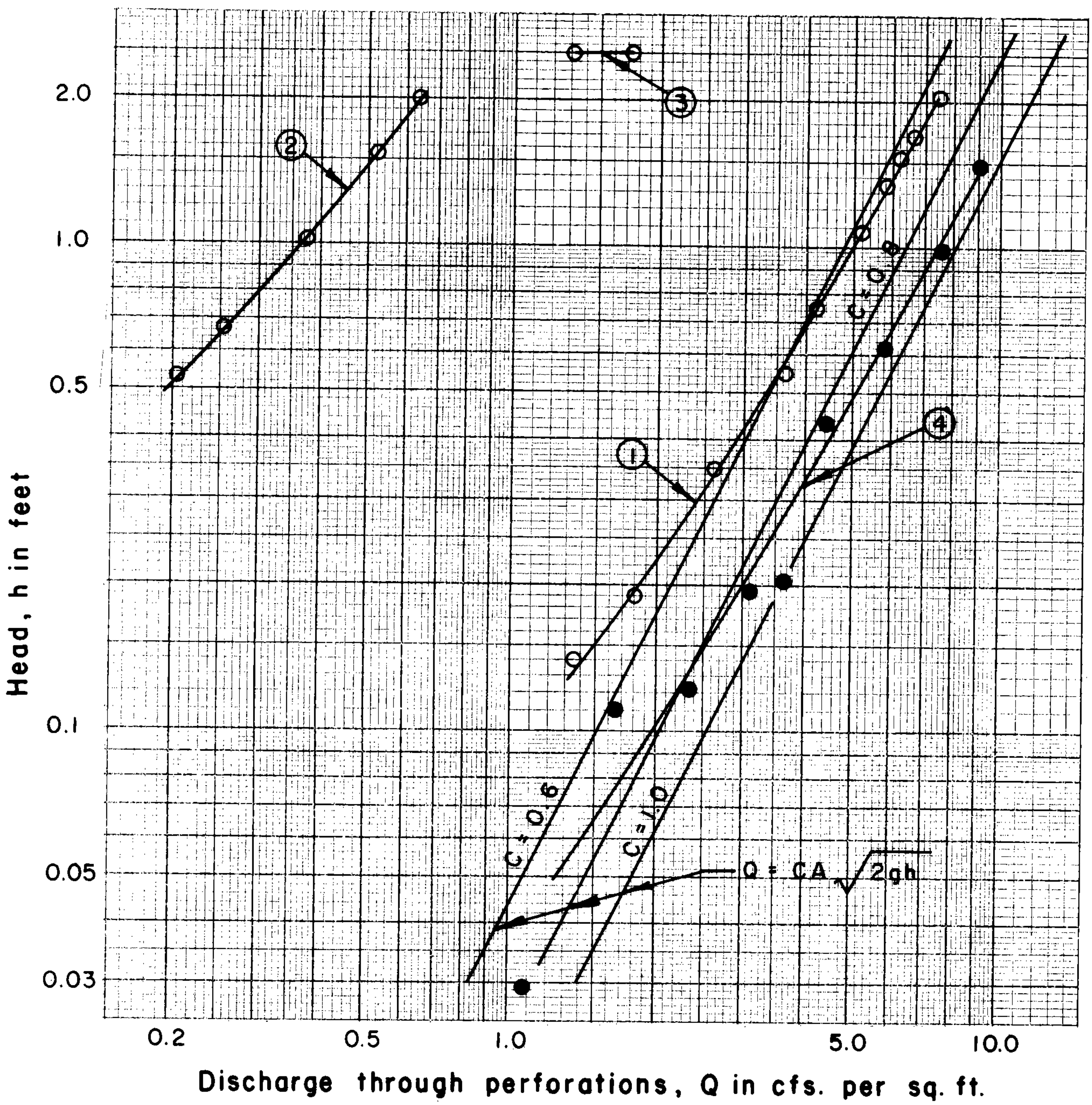
1. A comparison of the curve for $Q = CA\sqrt{2gh}$, C = 0.6, and curve No. 1, pipe in water only, indicates that a coefficient of discharge of 0.6 is reasonable for the perforations in this uncoated corrugated metal pipe.

2. With the uncoated corrugated metal pipe imbedded in medium SP, curve No. 2, discharge through the perforations is about 10% of the discharge without sand around the pipe.

3. The range represented by No. 3 shows that discharge through perforations of this coated corrugated metal pipe placed in coarse SP is about 20% of the discharge with the pipe in water only and assuming that C = 0.6.

4. Curve No. 4 is for flow into clay pipe with a wall thickness of 5/8 in. The perforation length to diameter ratio is 2.5, in the range of short tubes, where a discharge coefficient of 0.8 is normal. Flow through joints could not be separated from flow through perforations and it is not known how this would affect the discharge coefficient. Even assuming that C = 1.0, discharge through openings in this pipe placed in GP is greater than 80% of that without restriction.

In a recent study, "Laboratory Tests of Relief Well Filters", Report No. 1, MP S-68-4, Waterways Experiment Station, Corps of Engineers, two clean sands and a fine gravel were placed around a wood screen having 3/16 in. (4.76 mm) slots. Discharges were measured before and after surging and the unclogged slot area was determined by observation after testing. The D_{50} size of the finer sand was 2.7 mm. and the unclogged slot area was 20% of the total. The coarser sand had a D_{50} size of 3.6 mm. and an unclogged slot area of 50%. The gravel had a D_{50} size of 4.7 mm. (about the slot width) and an unclogged slot area of 70%.



1 From First Progress Report on Performance of Filter Materials, J. C. Gillou, Univ. of Ill., 1960. 8 in. dia. cmp., 5/16 in. perforations, assuming 16 per foot. Pipe in water only.

2 Same as 1 but with pipe in medium SP, gradation 1, Fig. A-2.

3 From WES TM 183-1, 1941. 6 in. dia. coated cmp., effective perforation dia. 3/16 in., 40 per foot. Pipe in coarse SP, gradation 2, Fig. A-2.

4 From Spindletop Research Report 580, 1967. 6 in. dia. clay pipe, 1/4 in. perforations, 44 per 3 ft. length, wall thickness 5/8 in. Pipe in GP equivalent to Indiana No. 7 Stone, gradation 3, Fig. A-2. Joints considered as perforations for curve.

Figure A-1. Head-discharge relationship for pipe perforations.

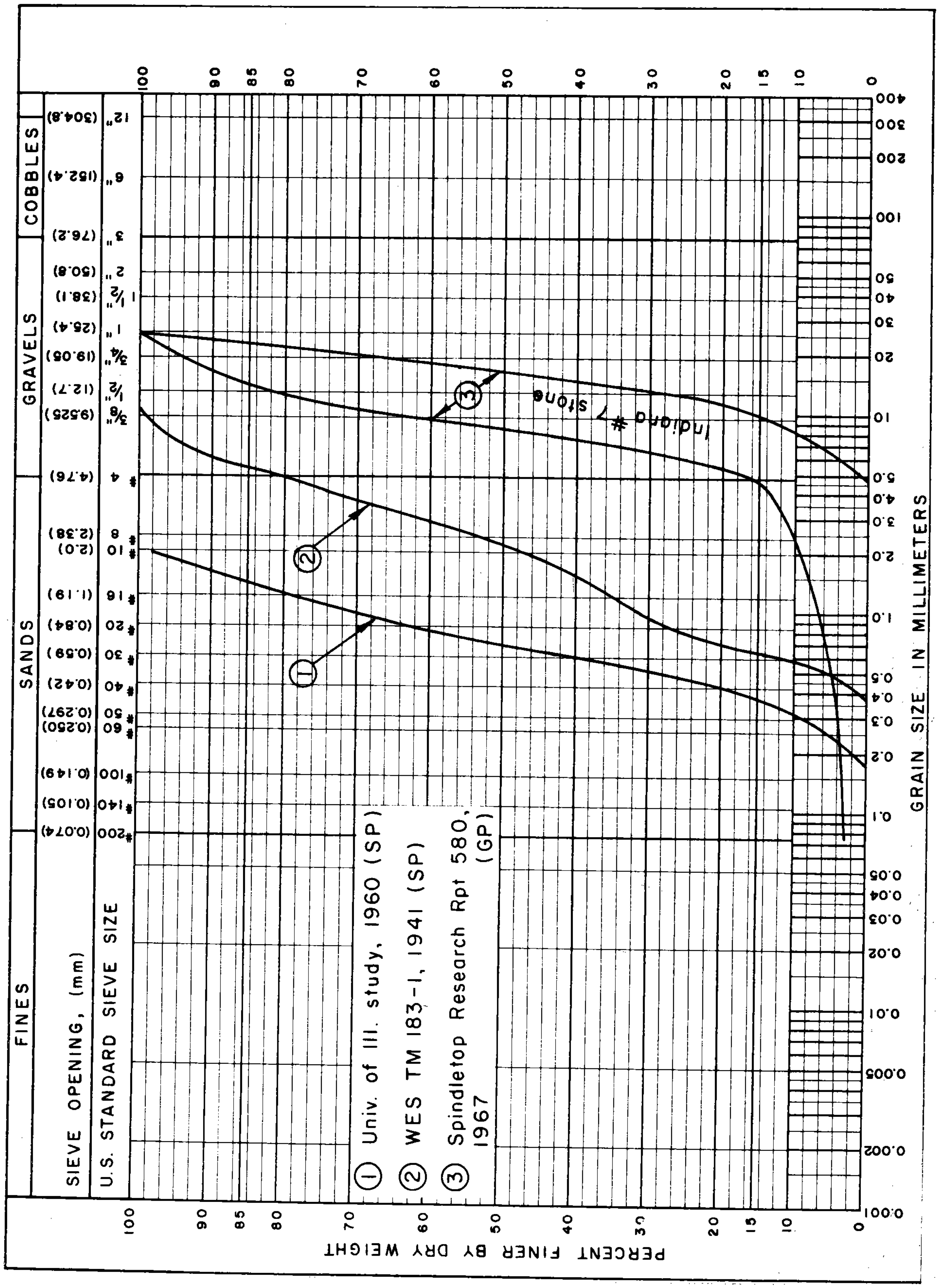


Figure A-2. Gradation of drain material used in the studies from which Fig. A-1 curves were developed.

Considering that only a few studies are available for this type of review and that these are not complete in every aspect, any procedure developed for estimating discharge into perforated pipe must necessarily be conservative. The above studies show that sands are more restrictive to flow through small openings than gravels. Therefore, the development that follows is limited to pipes placed in <u>gravel</u> drain material meeting the requirements that: (1) it will be <u>virtually</u> clean, (2) it will have a coefficient of uniformity less than 3, and (3) it will have a median or D_{50} size equal to or greater than the perforation diameter or slot width. Area or discharge reductions are made for conservatism: 70% for circular perforations and 40% for rectangular slots.

The area (A) per foot of pipe is given in Figure A-3 for 1/4 in., 5/16 in., and 3/8 in. diameter perforations. Flow quantity (q in cfs.) per foot of pipe can be estimated from Figure A-4 for circular perforations and from Figure A-5 for rectangular slots. The maximum orifice head considered is 2.0 feet since it is preferred that the water surface be maintained within the gravel drain material.

The flow equation for Figures A-3 and A-4 is:

$$q = CA_e\ (2gh)^{1/2} \quad \text{where}$$

q = discharge in cfs. per foot length of pipe

C = orifice coefficient (0.6 for circular perforations and 0.67 for rectangular slots).

A_e = effective area of openings per foot length of pipe (0.3A for circular perforations and 0.6A for rectangular slots, A being the non-restricted area). This correction is to account for blockage of openings by sand and gravel particles.

Note: Computations for discharge quantity curves included a conversion from square inches to square feet.

h = head over the orifice in feet.

ES-97 of NEH Section 5, Hydraulics, is recommended for estimating flow conditions within the pipe.

When high design discharges are involved and multiple outlets are not practical, more than one perforated pipe may be used to satisfy either the inflow (orifice) condition or the pipe flow condition.

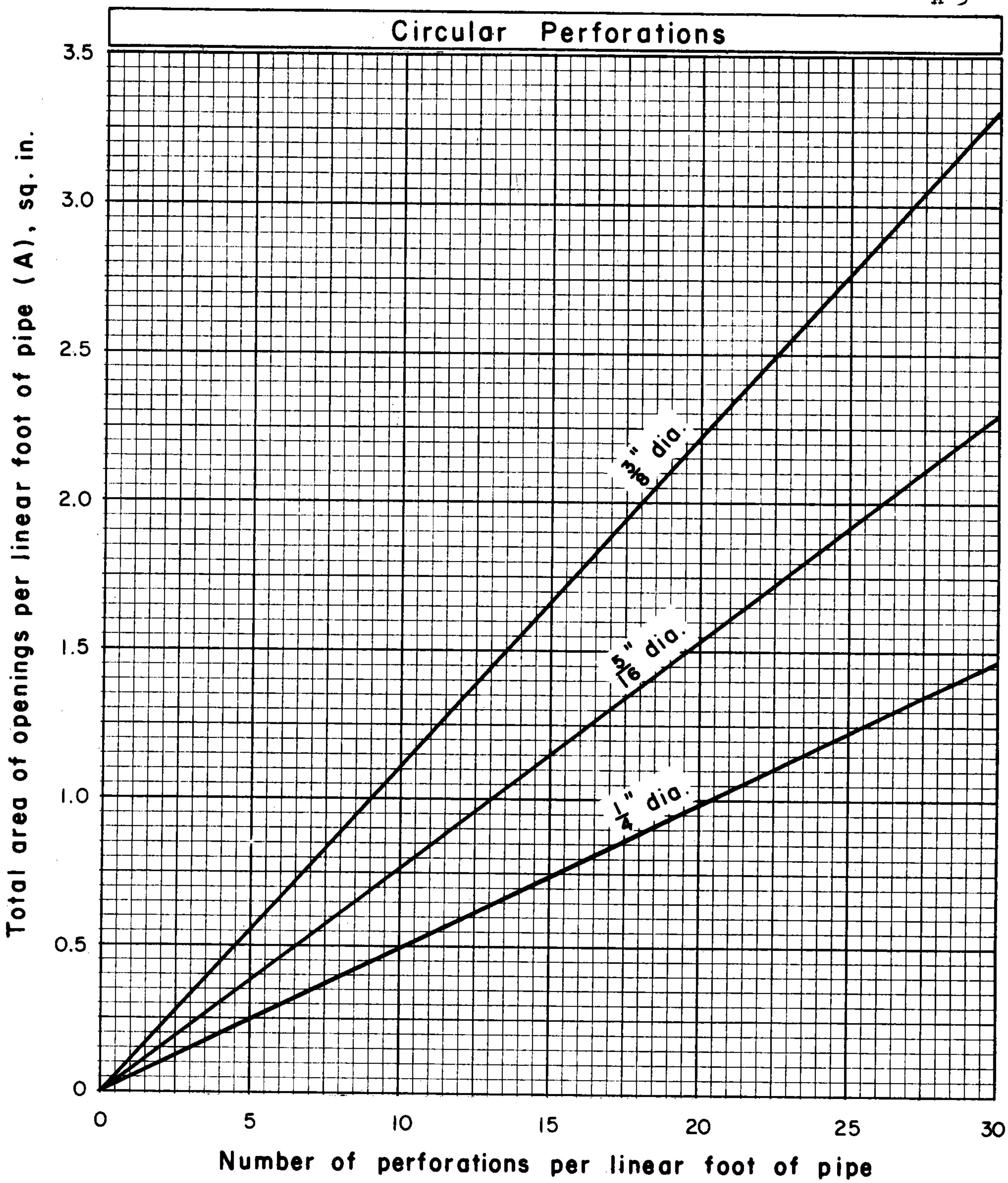


Figure A-3: Total area of circular perforations per foot length of pipe.

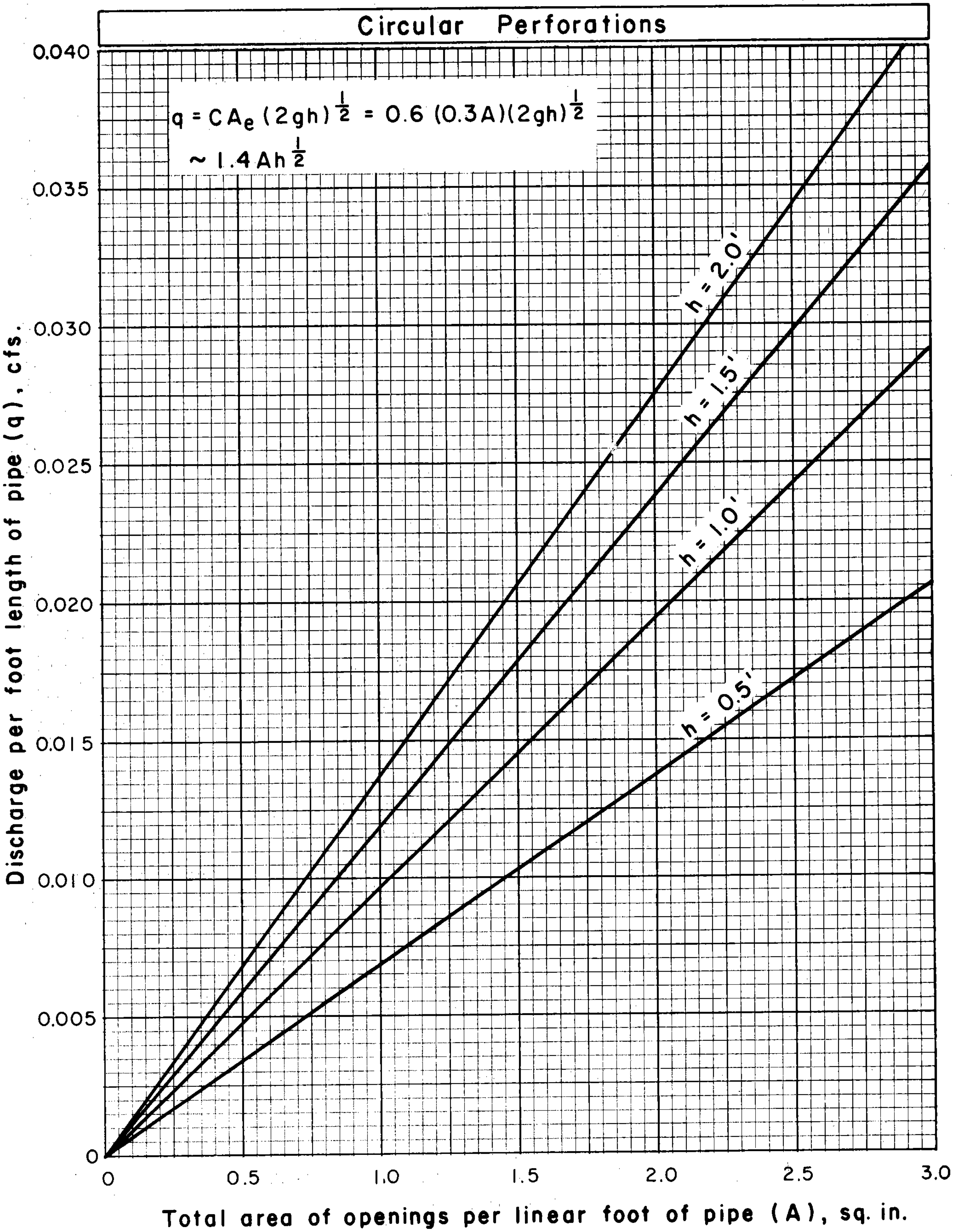


Figure A-4: Flow into pipe with circular perforations.

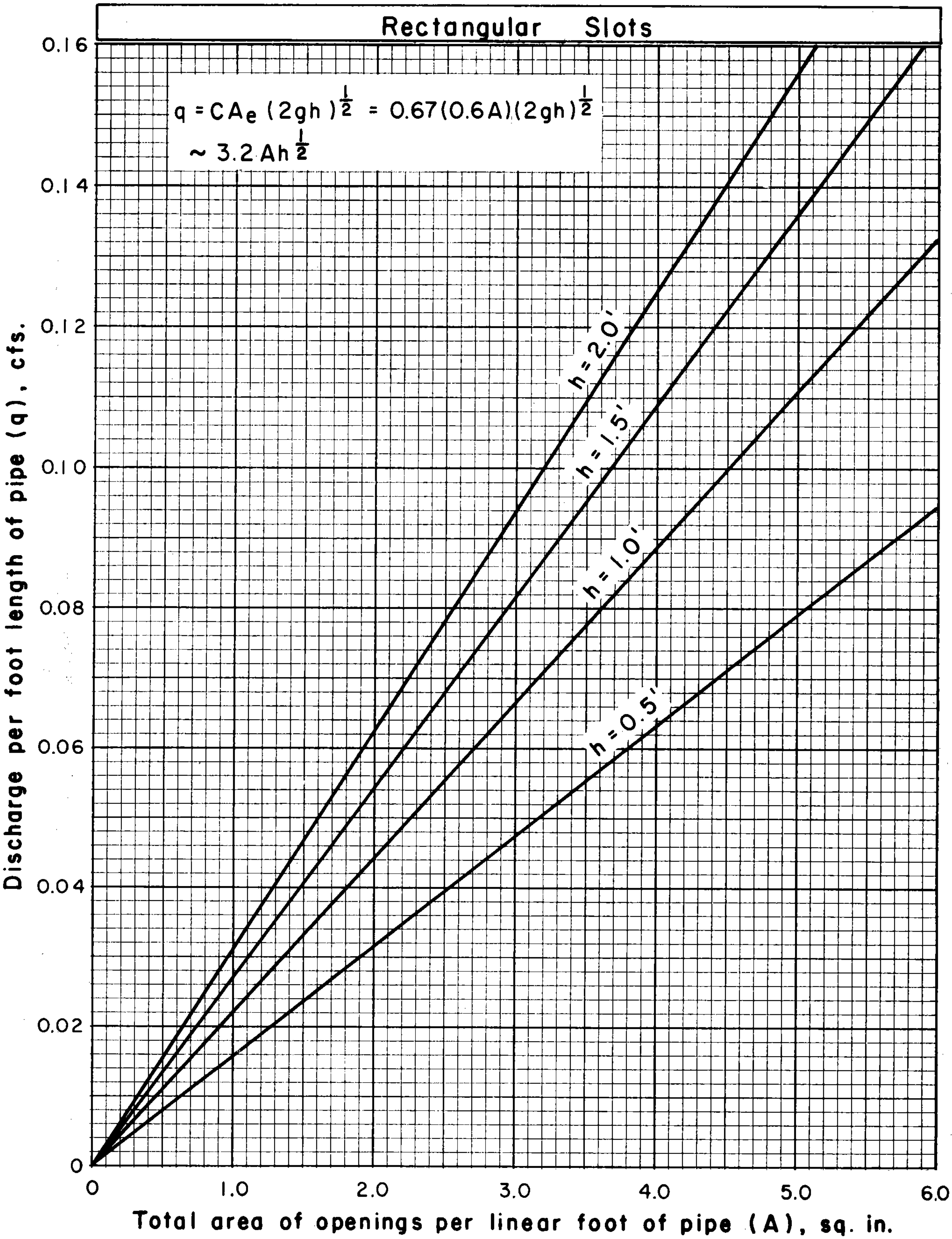


Figure A-5: Flow into pipe with rectangular slots.

Appendix B

Relief Wells

I. General

A simplified and approximate method for design of relief wells is given in this appendix. It is based on well formulae developed for confined or artesian aquifers that are homogeneous and isotropic. Refer to the work of C. I. Mansur and R. I. Kaufman as edited by G. A. Leonards in "Foundation Engineering", 1962, McGraw-Hill Book Company, Inc., page 281.

In this appendix, a blind well refers to a relief well which consists solely of either drain material or drain material and filter material, i.e. it has no well screen or pipe. A fully penetrating well is one in which the well extends entirely through the aquifer, whereas a partially penetrating well extends into the aquifer but not entirely through it.

Head lost in flow from the reservoir to the free outlet is divided into three parts: H, H_m, and H_w. Many symbols and definitions are given in Figure B-1; other symbols are defined where they are first used.

A. H is the head loss in the aquifer to a point midway between wells.

$$H = h_e - h_m = \frac{Q_w}{k_f D}\left(\frac{L_e}{a} - 0.11\right) \qquad \text{(Eq. B-1a)}$$

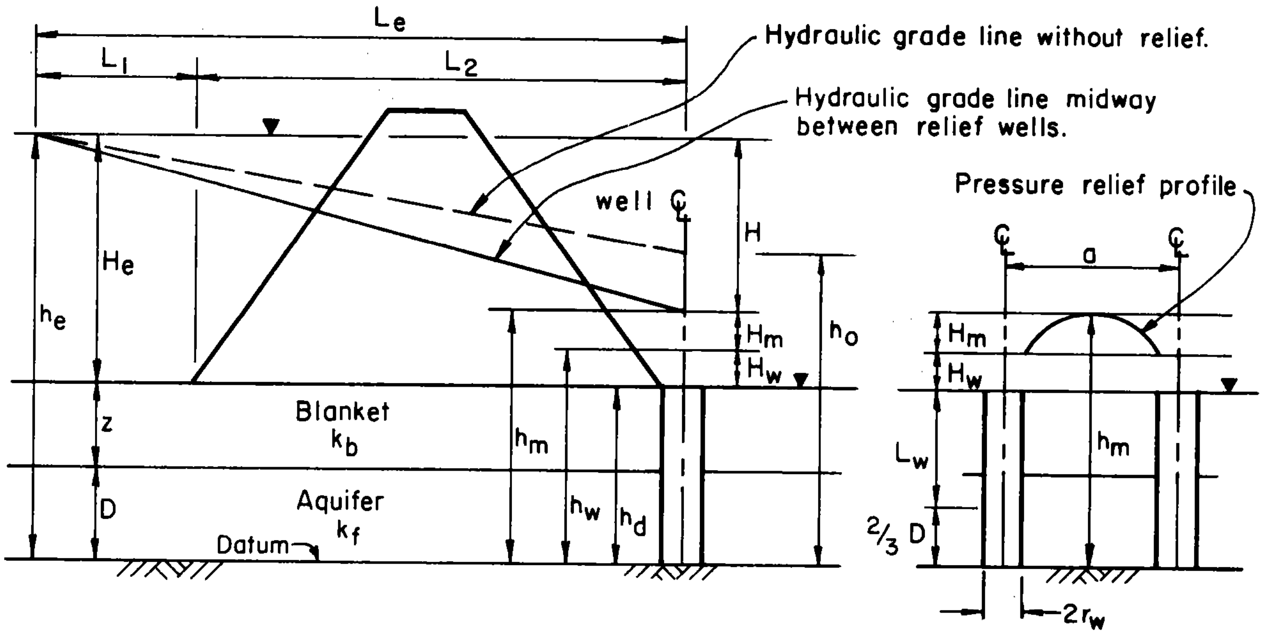
This is simplified by dropping the term 0.11.

$$H = \frac{Q_w L_e}{k_f D a} \qquad \text{(Eq. B-1b)}$$

This head loss depends upon the uplift pressure that can be tolerated midway between wells near the downstream toe of an embankment.

B. H_m is the head loss in the aquifer from a point midway between wells to a well.

$$H_m = h_m - h_w = \frac{Q_w}{2\pi k_f D}\ \ln\left(\frac{a}{\pi r_w}\right) \qquad \text{(Eq. B-2)}$$



A_w = cross sectional area of well pipe or well drain material
a = spacing of wells
D = thickness of aquifer
H = h_e - h_m = head loss associated with flow Q_w to h_m
H_e = h_e - h_d = potential head between reservoir water surface and well discharge height
H_m = h_m - h_w = head loss associated with flow Q_w between h_m and h_w
H_w = h_w - h_d = head loss associated with flow Q_w from each well
h_e = height of reservoir water surface above datum
h_d = height of well discharge above datum
h_o = height of hydraulic grade line above datum at downstream toe of embankment without wells
h_m = height of hydraulic grade line above datum at mid-point between installed flowing wells
h_w = height of piezometric surface above datum at effective diameter of well
k_b = permeability coefficient of blanket (vertical)
k_f = permeability coefficient of aquifer (horizontal)
k_w = permeability coefficient of drain material in well
L_w = average vertical seepage length in well
L_1 = effective length of upstream blanket
L_2 = length of embankment base
L_e = L_1 + L_2
Q_w = quantity of flow to well
$2r_c$ = diameter of inner well core or diameter of well pipe
$2r_h$ = diameter of drill hole for well
$2r_w$ = effective diameter of well
z = thickness of blanket

Figure B-1. Relief well design, symbols

Substituting the expression for Q_w from Eq. B-1b into Eq. B-2 gives:

$$H_m = 0.3665a \left(\frac{H}{L_e}\right) \log_{10}\left(\frac{a}{\pi r_w}\right) \qquad \text{(Eq. B-3)}$$

Chart solutions of Eq. B-3 for fully penetrating wells having effective well diameters of 24, 20, 16, 12 and 10 inches are given in Figures B-2 through B-6 for various values of H/L_e and a.

Effective well diameter ($2r_w$) is defined as follows:

For well screen without filter (naturally developed filter), $2r_w$ = outside diameter of well screen ($2r_c$).

For well screen with filter, $2r_w = 0.5$ (outside diameter of filter + diameter of well screen) = $0.5\ (2r_h + 2r_c)$.

For blind well consisting of drain material only -- no filter, $2r_w$ = diameter of drain material ($2r_h$).

For blind well consisting of drain material and filter material, $2r_w = 0.5$ (outside diameter filter + diameter of drain material) = $0.5\ (2r_h + 2r_c)$.

C. H_w is the sum of all head losses in a well.

For blind wells, $H_w = \dfrac{Q_w L_w}{k_w A_w}$ (Eq. B-4)

For wells with screens and riser pipe, H_w is the sum of screen losses, pipe friction, fitting losses, and velocity head.

The sum of H, H_m, and H_w equals the total net head, H_e, available for flow.

$$H_m + H_w = H_e - H$$

From Figure B-1, $H_e - H = h_m - h_d$

$$\therefore H_m + H_w = h_m - h_d \qquad \text{(Eq. B-5)}$$

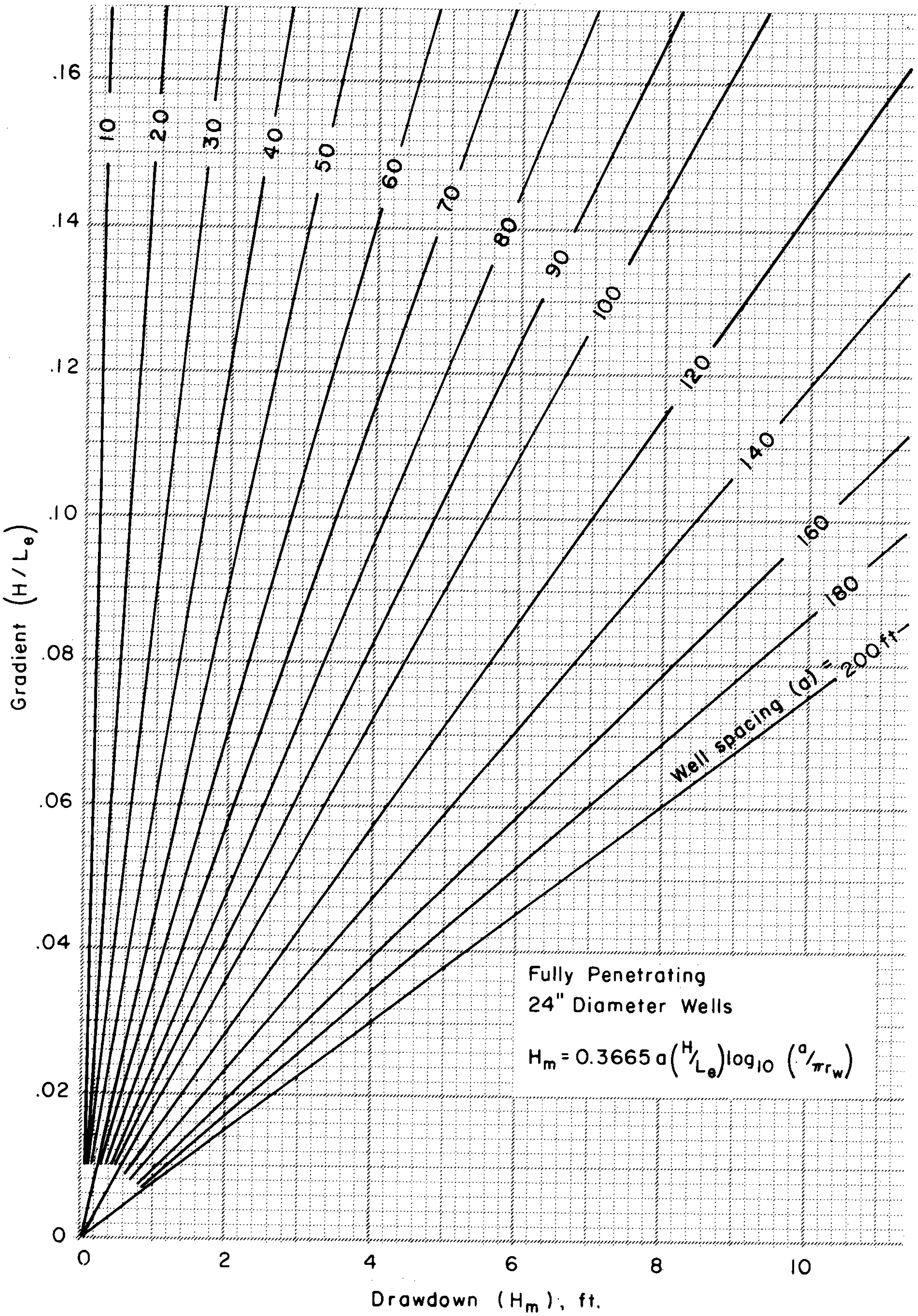


Figure B-2. Relief well design

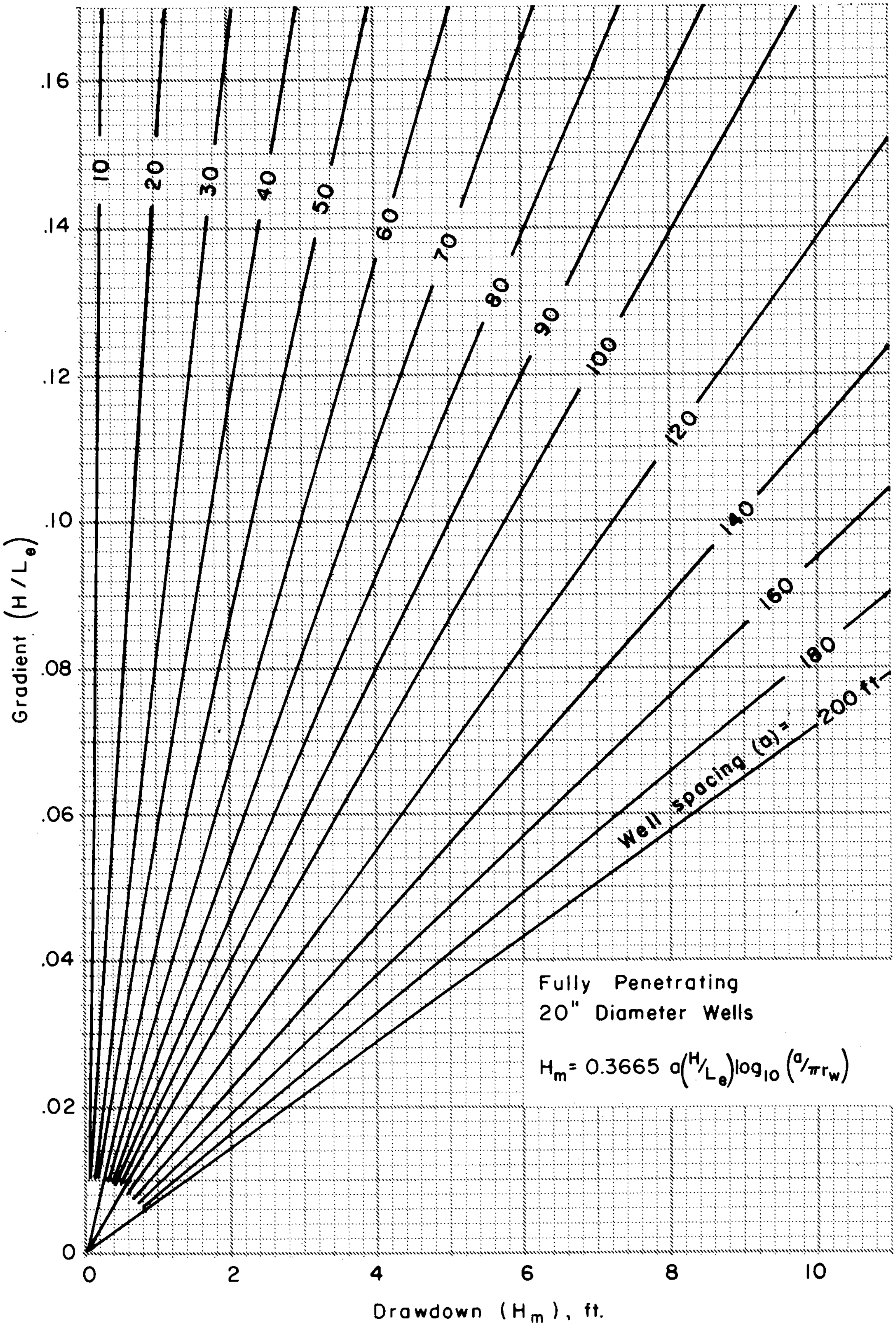


Figure B-3. Relief well design

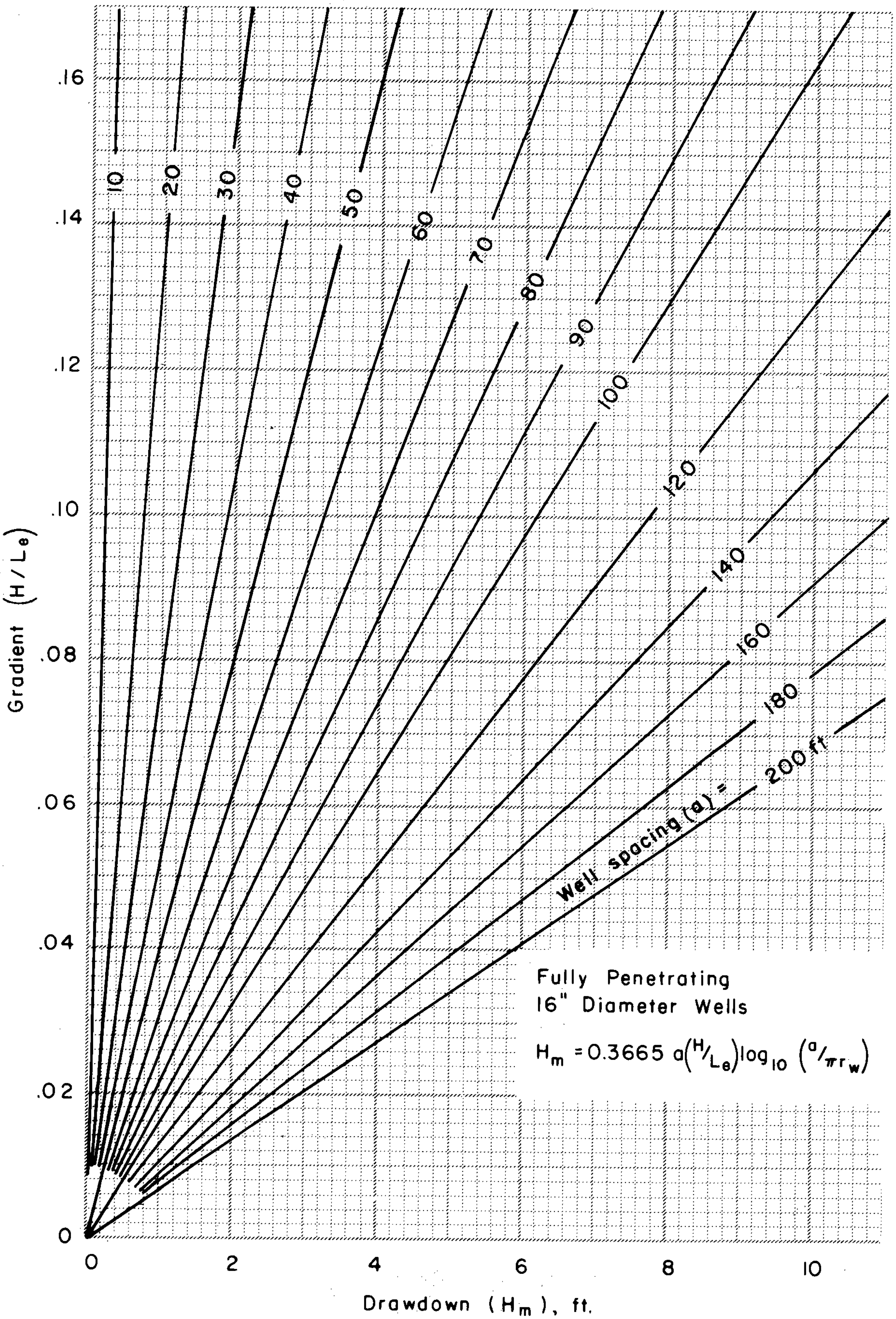


Figure B-4. Relief well design

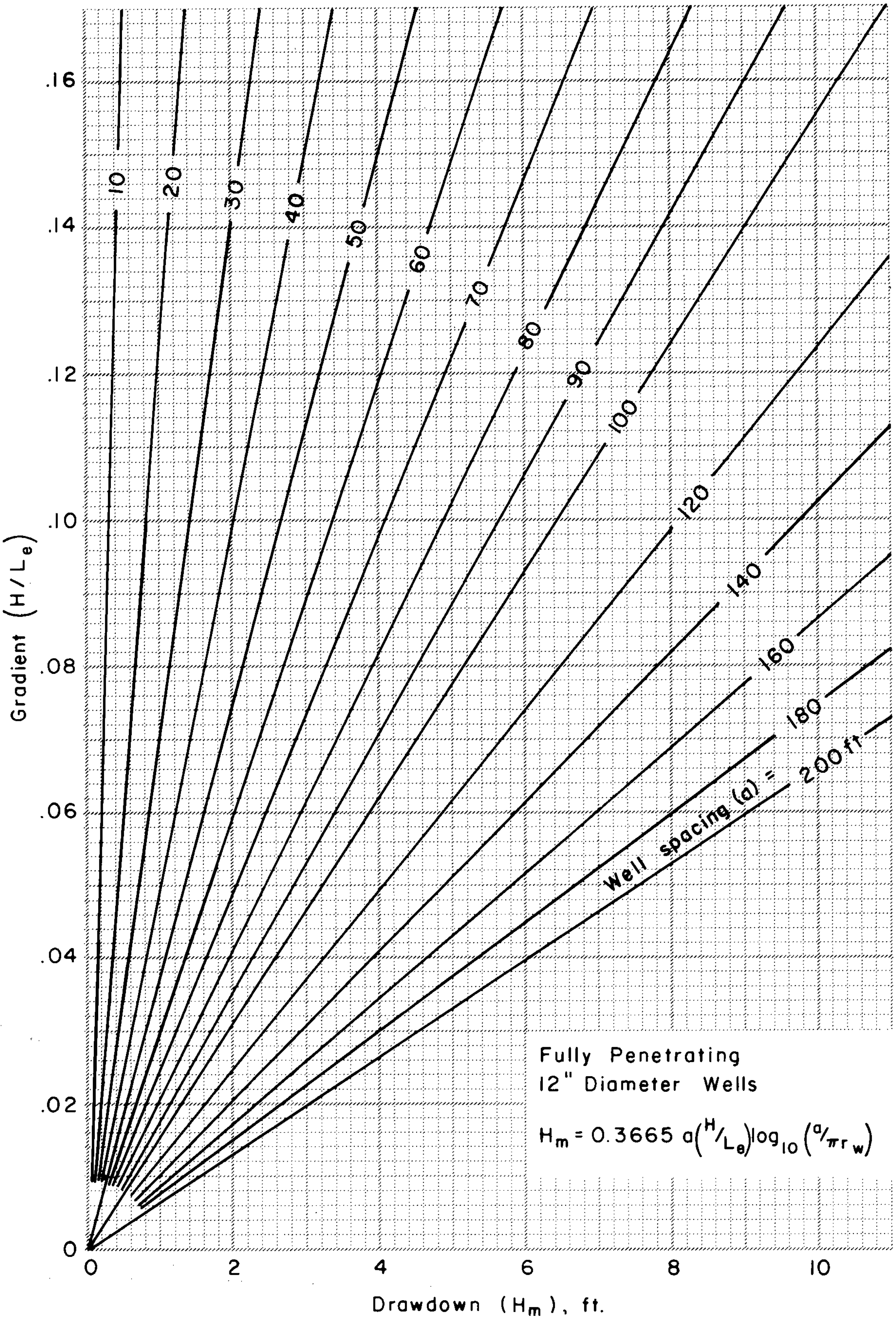


Figure B-5. Relief well design

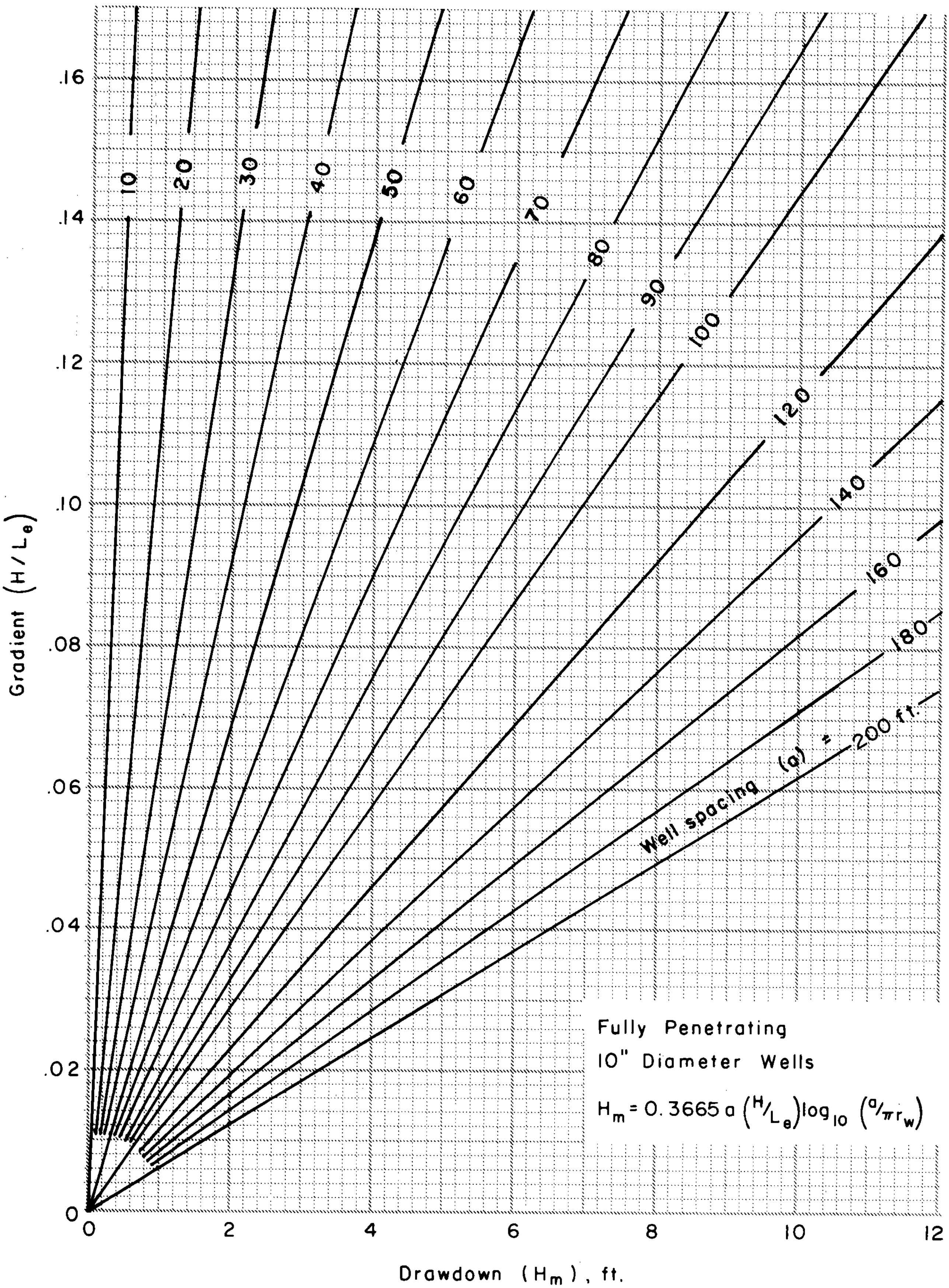


Figure B-6. Relief well design

II. Design Procedures

A. Fully penetrating blind wells

1. Determine $L_e = L_1 + L_2$, tolerable h_m, and $H = h_e - h_m$ from site conditions using methods similar to Bennett which are illustrated in the SM-10, Chapter 12, pages 12-19 through 12-21.

$$L_1 = \left|\frac{k_f}{k_b} zD\right|^{1/2} \qquad \text{(Eq. B-6)}$$

$$\underset{\text{(tolerable)}}{h_m} = \frac{z\ \gamma_{sub}}{F_h\ \gamma_w} + (z + D) \qquad \text{(Eq. B-7)}$$

where γ_{sub} = submerged unit weight of blanket material
γ_w = unit weight of water
F_h = factor of safety relative to heaving of blanket midway between wells (≥ 1.5)

2. Compute Q_w in terms of a from Eq. B-1b.

3. Compute L_w

$$L_w = h_d - \frac{2D}{3} \qquad \text{(Eq. B-8)}$$

4. Solve for H_w in terms of a, substituting Q_w from step 2 into Eq. B-4.

5. Plot the value for h_d as shown in Figure B-7.

6. For two assumed values of a, plot the straight line $h_w = (h_d + H_w)$ vs. a as shown in Figure B-7.

7. Plot the value for h_m as shown in Figure B-7. This may be the tolerable value from step 1 or a lesser value.

8. Determine values of H_m for various well spacings using the appropriate set of curves (Figures B-2 through B-6). Effective well diameter and ratio H/L_e are known.

   Plot the curve $h_w = (h_m - H_m)$ vs. a as shown in Figure B-7.

9. The intersection of the two curves gives the well spacing at which Eq. B-5 ($H_m + H_w = h_m - h_d$) is satisfied.

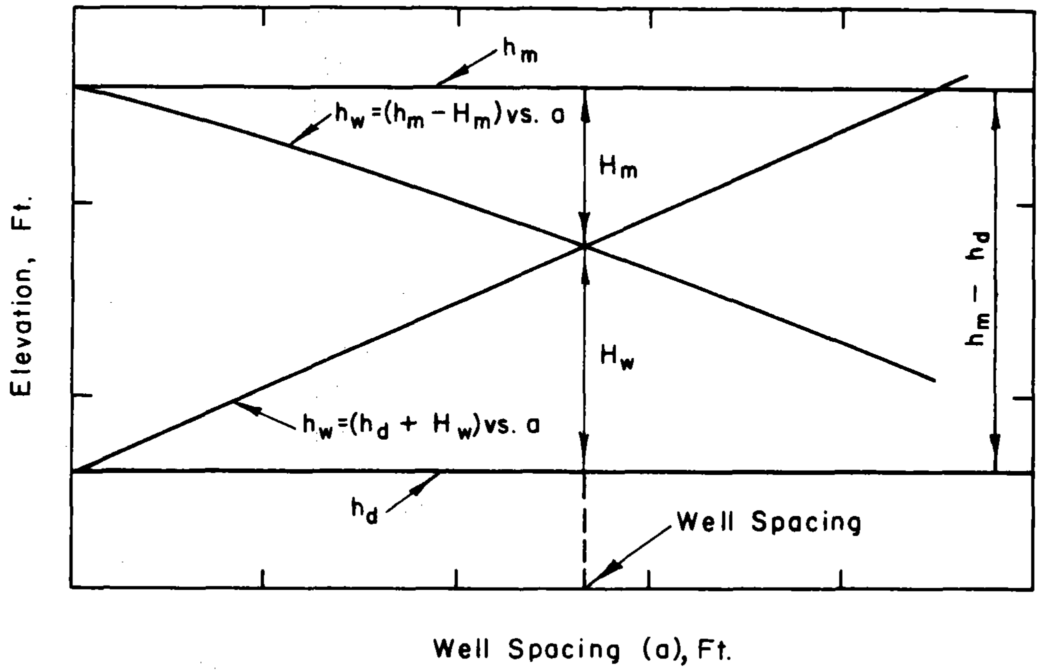


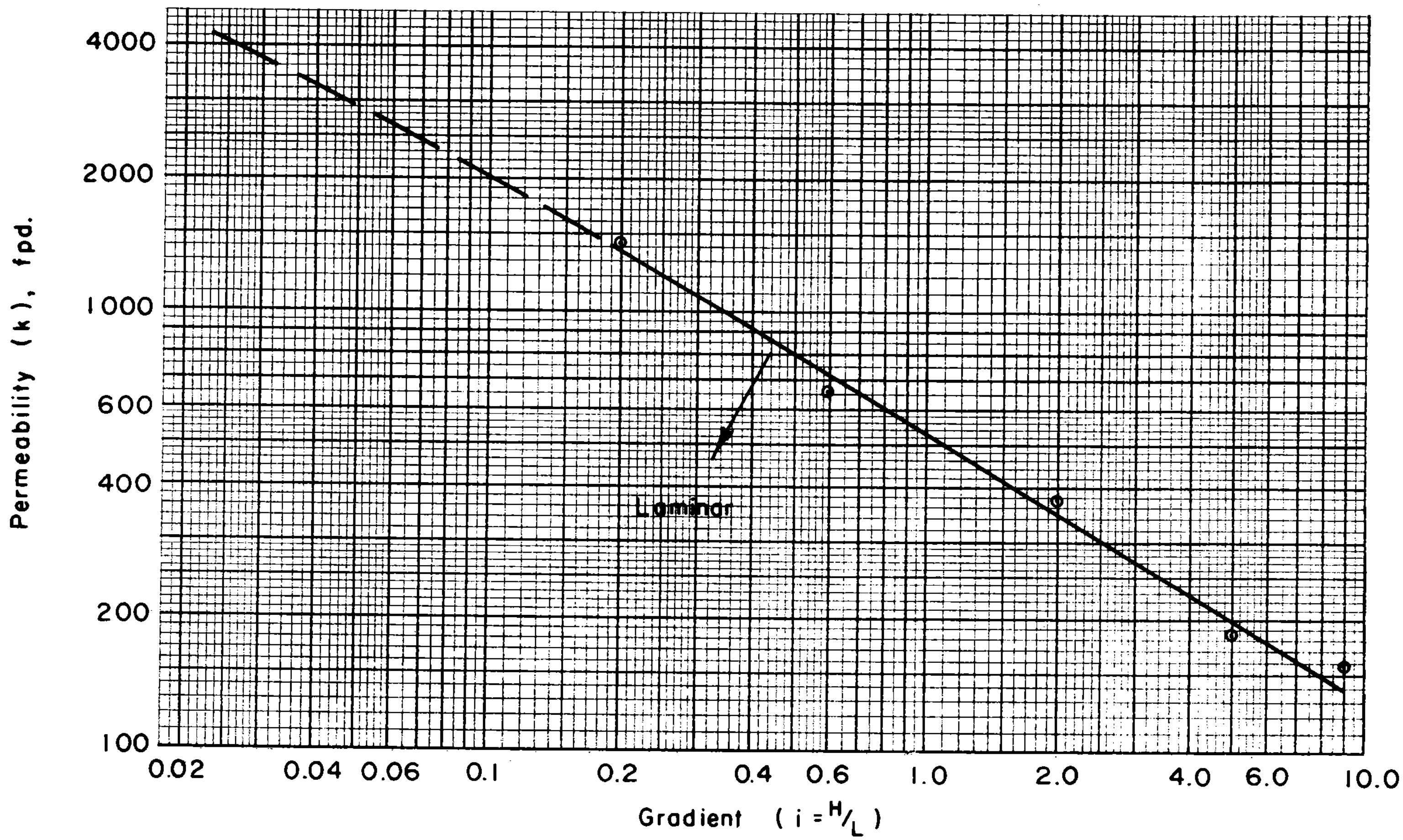
Figure B-7. Head loss vs. well spacing (arithmetic scales)

10. Use of Darcy's law in Eq. B-4 for well loss, H_w, depends on the existence of laminar flow conditions in the well. Check this by plotting k_w vs. i_w in Figure B-8. If the plotted point falls outside the laminar region, adjust the well spacing, the discharge elevation, or the well diameter and re-do the previous steps.

    An alternate to this requirement is developing, by test, a curve of unit discharge vs. gradient for the drain material to be used. From the known values of Q_w/A_w (Q_w in terms of a) and L_w, H_w can be determined from the test curve for various spacings, a, and entered into step 6. ($H_w = i_w L_w$)

B. Partially penetrating blind wells.

Where blind wells partially penetrate homogeneous and isotropic aquifers, the following equation is applicable. See "Groundwater and Seepage" by M. E. Harr, 1962, McGraw-Hill Book Co., page 263.



This chart, which defines a limiting velocity for laminar flow below which Darcy's law is valid, was developed from data of Table 3, p. 256, vol. 26, No. 12, Public Roads, "Highway Subdrainage," by Barber and Sawyer.

More reliable permeability rates can be determined by making permeability tests of the specified drainage material at the designed gradient.

Figure B-8. Limiting gradient for laminar flow at various k values

$$G = \frac{w}{D}\left|1 + 7\left(\frac{r_w}{2w}\right)^{1/2}\cos\left(\frac{\pi w}{2D}\right)\right| \qquad \text{(Eq. B-9)}$$

where w = depth of penetration of well into aquifer and

G = the ratio $\frac{Q \text{ partially penetrating}}{Q \text{ fully penetrating}}$ for the same value of H_m

Figure B-9 is a solution of this equation.

The procedure is the same as for fully penetrating blind wells except that L_w is computed by Eq. B-10 below and the points $h_w = h_m - H_m$ are plotted vs. Ga, a being the spacing determined for full penetration. The intersection of $(h_d + H_w)$ vs. a and $(h_m - H_m)$ vs. Ga gives the spacing corrected for partial penetration.

$$L_w = h_d - (D - w) \qquad \text{(Eq. B-10)}$$

C. Fully penetrating wells with screens and riser pipe.

The procedure is the same as for fully penetrating blind wells except that well loss, H_w, is determined by summation of screen loss, pipe friction loss, fitting or coupling losses, and velocity head loss.

1. Screen loss. It has been determined by test and experience that screen friction loss can be neglected if the entrance velocity is 0.1 fps or less. Refer to "Ground Water and Wells", 1966, Edward E. Johnson, Inc., page 193.

   Estimate the entrance velocity by dividing Q_w by 0.6 of the unclogged area of the screen. The 0.6 factor is introduced for this estimate similar to the requirement suggested for rectangular slotted pipe in Appendix A. Most screen manufacturers will provide information on the total opening area for their various screens.

2. Pipe friction loss, H_f, may be estimated from Figure B-10. When Hazen-Williams roughness coefficient (C) = 100, obtain friction head loss in 100 feet of well pipe (H_{f100}) directly from Figure B-10. Then, friction loss for actual length of pipe $(H_f) = (L_w/100)(H_{f100})$. When C has a value other than 100, use line in upper right-hand corner of Figure B-10 to obtain factor F. Then, friction loss in actual length of well pipe $(H_{fc}) = (L_w/100)(H_{f100})(F)$. Other methods of determining H_f are presented in SCS NEH Section 5, Hydraulics.

3. Velocity head loss, H_v, may be estimated from Figure B-11.

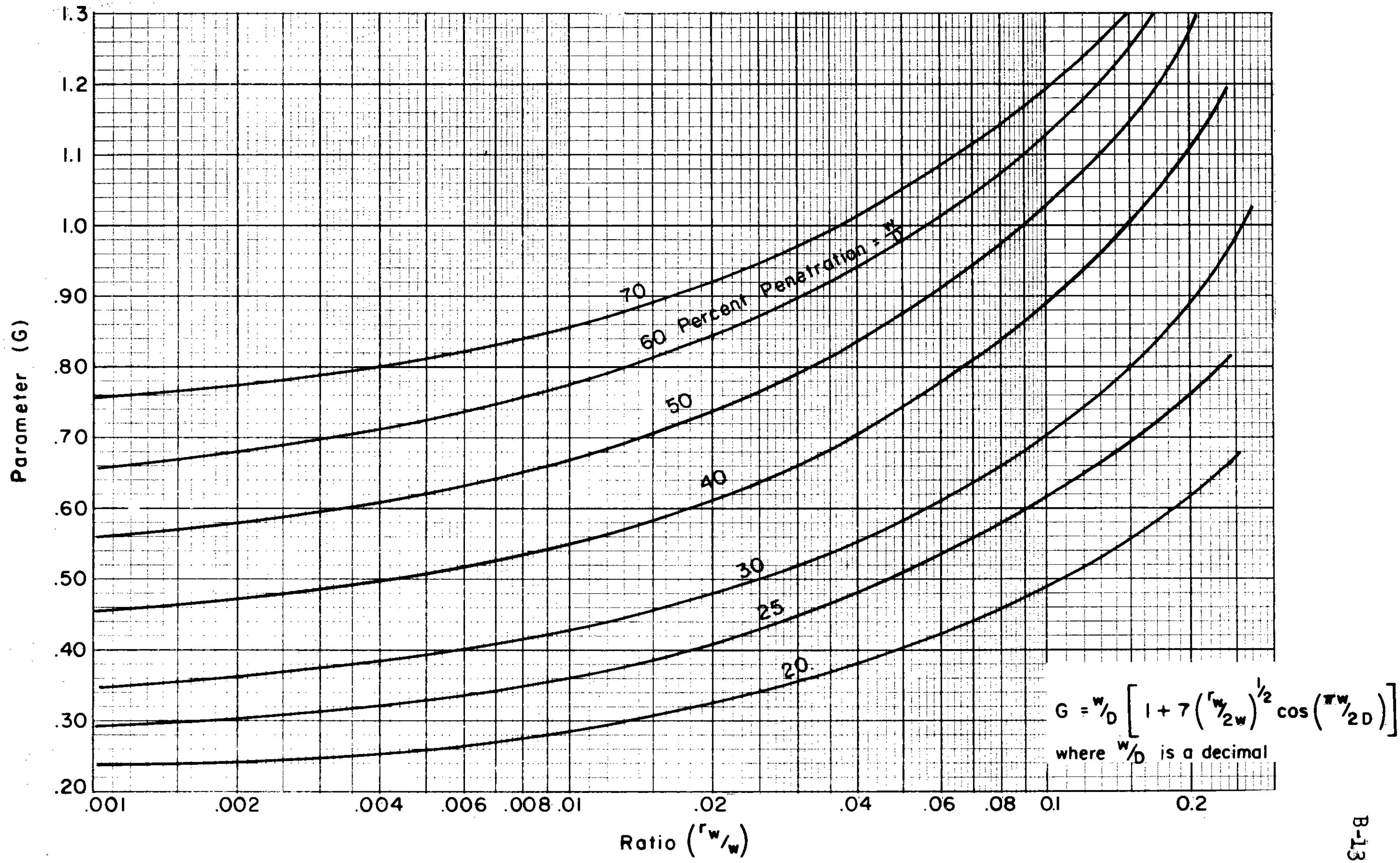


Figure B-9. Parameter G vs. r_w/w and percent penetration for partially penetrating wells with open bottom

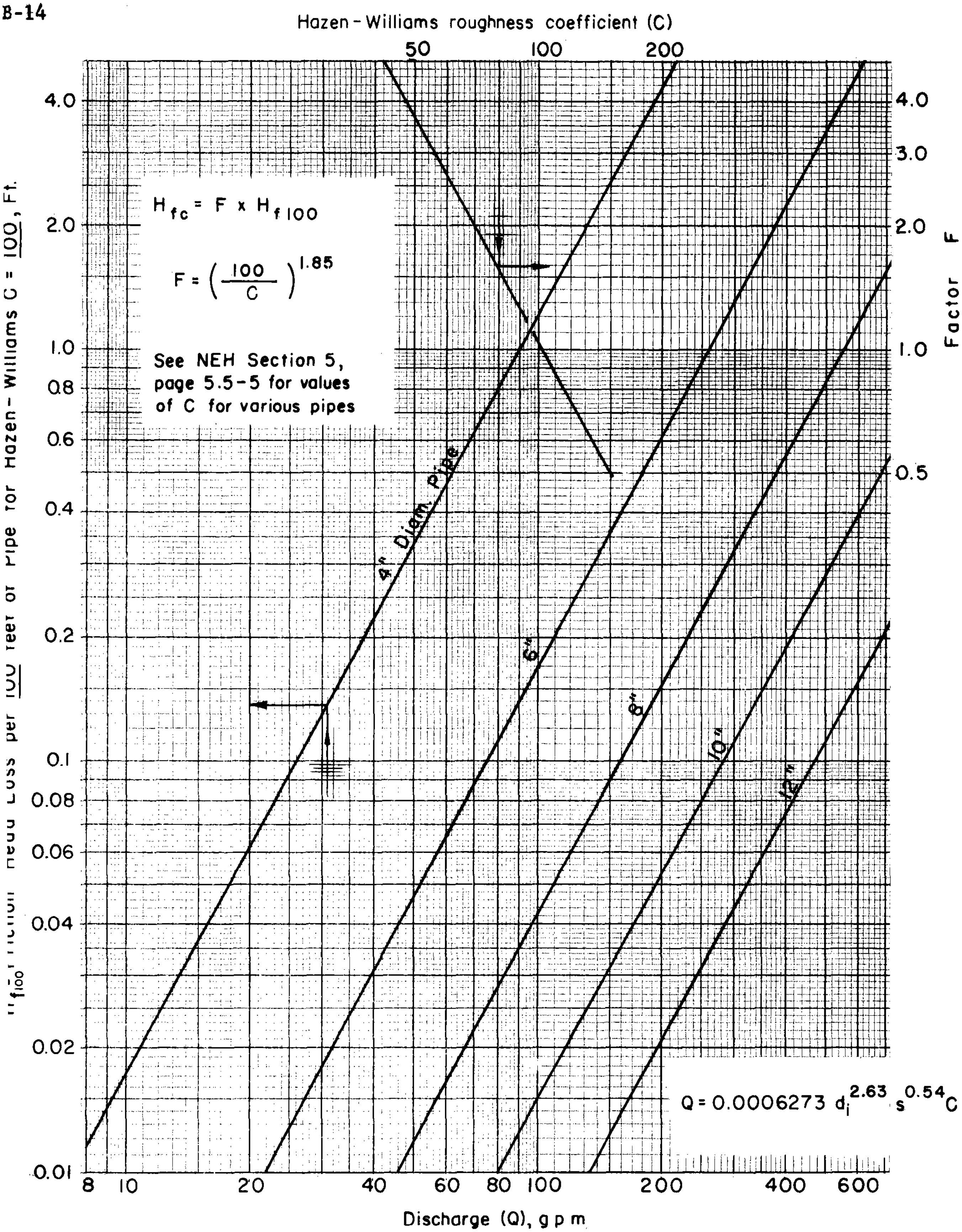


Figure B-10. Friction head loss for pipe

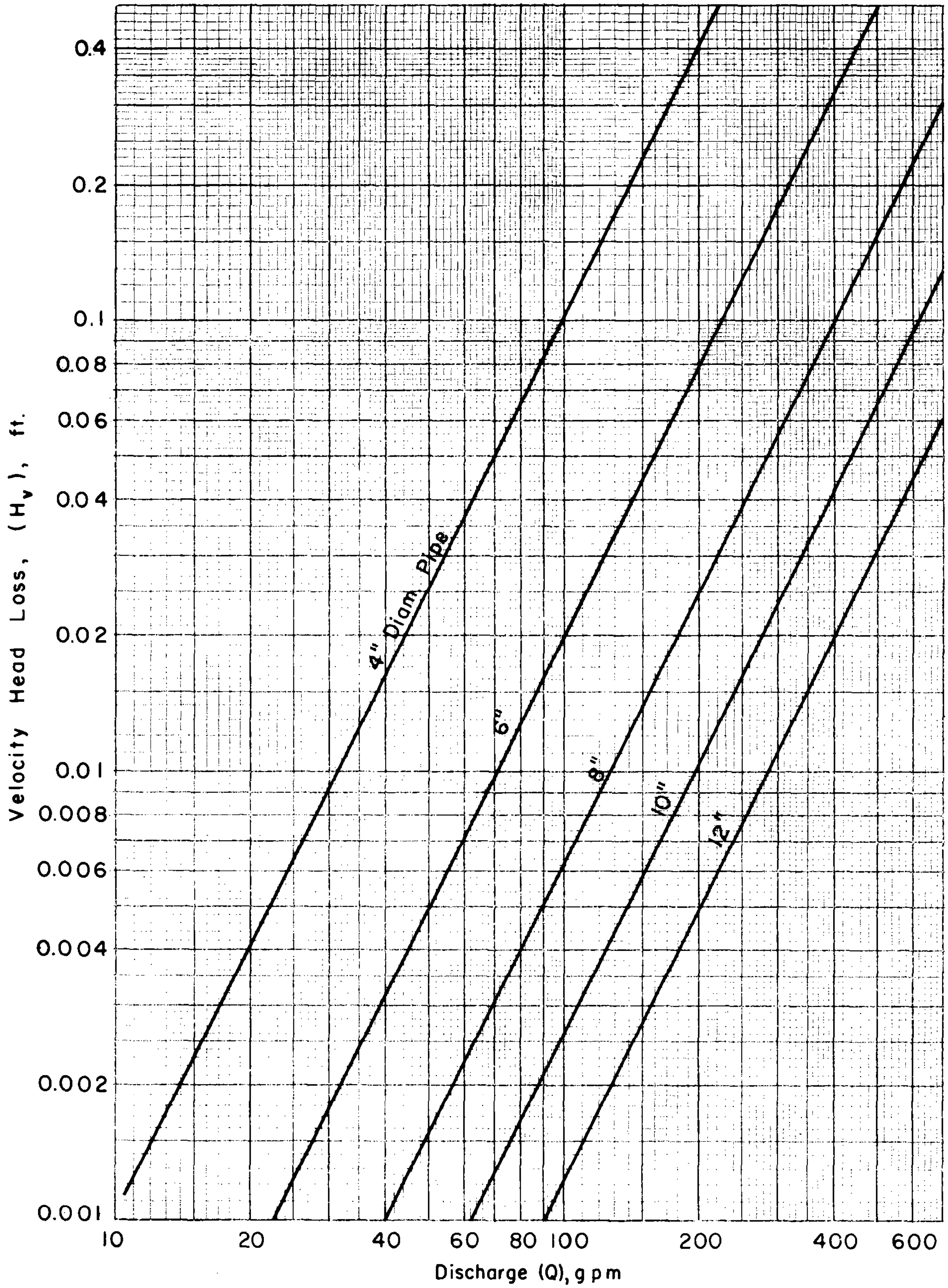


Figure B-11. Velocity head loss

4. Multiply the number of connections by 1.5 H_V to determine connection losses, H_X.

An alternate to this method is given in the Corps of Engineers, EM-1110-2-1903, 1963.

D. Partially penetrating wells with screen and riser pipe.

It is recommended that procedures outlined in the Corps of Engineers EM 1110-2-1903, 1963, be used.

III. Application notes

A. The formulae presented apply to confined aquifers that are essentially homogeneous and isotropic. Aquifers are generally stratified, making it necessary to transform layer thicknesses and permeabilities to an equivalent isotropic section before entering the formulae. An excellent discussion of stratified aquifers and transformation is made by W. J. Turnbull and C. I. Mansur, "Relief Well Systems for Dams and Levees", ASCE Transactions, Vol. 119, 1954, pages 842-878, and in the accompanying discussion by P. T. Bennett.

B. Filter and drain material must meet gradation requirements for prevention of piping.

C. It is suggested that screen slot width be the same as or smaller than the D_{50} size of surrounding drain material.

Reference List

1. Barber, E. S., and Sawyer, C. L., "Highway Subdrainage", Public Roads, Vol.26, No. 12, 1962.

2. Bennett, P. T., "The Effect of Blankets on Seepage Through Pervious Foundations", ASCE Transactions, Vol. 111, 1946.

3. Casagrande, A., "Seepage Through Dams", Contributions to Soil Mechanics, Boston Society of Civil Engineers, 1925-1940.

4. Cedergren, H. R., Seepage, Drainage, and Flow Nets, 1967, John Wiley and Sons, Inc.

5. Corps of Engineers, "Design of Finite Relief Well Systems", EM 1110-2-1905, 1963.

6. Corps of Engineers, "Laboratory Tests of Relief Well Filters", Report 1, WES MP 5-68-4, 1968.

7. Corps of Engineers, "Investigation of Filter Requirements for Underdrains", WES TM 183-1, 1941.

8. Gillou, J. C., "First Progress Report on Performance of Filter Materials", University of Illinois, 1960.

9. Ground Water and Wells, 1966, Edward E. Johnson, Inc.

10. Harr, M. E., Groundwater and Seepage, 1962, McGraw-Hill Book Company, Inc.

11. Mansur, C. I., and Kaufman, R. I., "Dewatering", Edited by Leonards, G. A., Foundation Engineering, 1962, McGraw-Hill Book Company, Inc.

12. Spindletop Research, Report 580, "A Comparison of Flow Properties of Perforated Clay Pipe and Plain Drain Tile in Subsurface Drainage", 1967.

13. Sherard, J. L., et al, Earth and Earth Rock Dams, 1963, John Wiley and Sons, Inc.

14. Turnbull, W. J., and Mansur, C. I., "Relief Well Systems for Dams and Levees", ASCE Transactions, Vol. 119, Paper 2701, 1954.

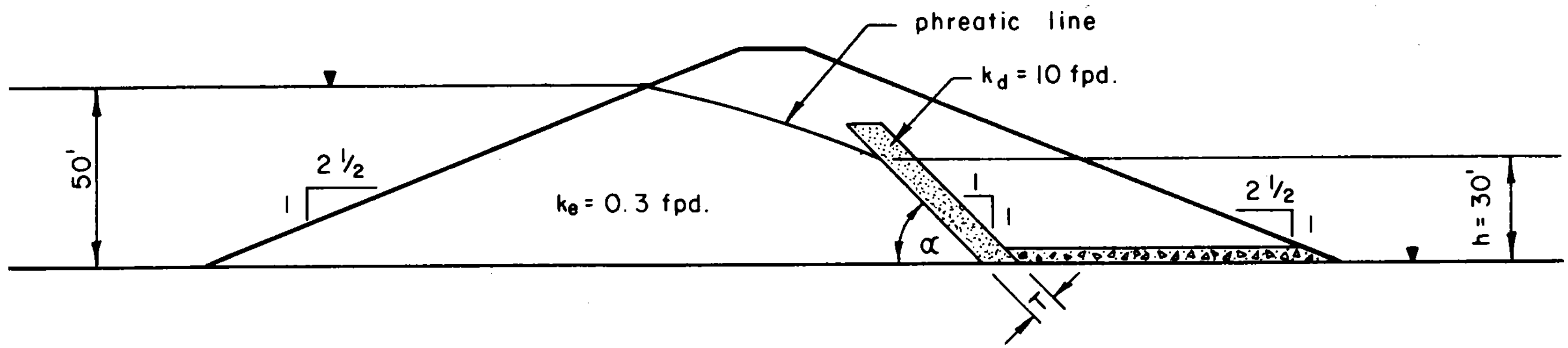
15. USDA, Soil Conservation Service, "Basic Soil Mechanics", SM-10 Manual, 1966.

16. USDA, Soil Conservation Service, National Engineering Handbook, Section 5, "Hydraulics".

Appendix C

Appendix C contains examples for the various drain types discussed.

Example C-1: Sloping embankment drain (embankment not susceptible to cracking).



Determine horizontal thickness of sloping embankment drain using Figure No. 1, Sec. IV, A. Outlet is adequate.

1. Locate egress point of phreatic line by flow net or A. Casagrande's methods (pages 12-6 through 12-10, SM-10, Basic Soil Mechanics, 1966). h = 30 ft.

2. $\frac{k_d}{k_e} = 33$, s = 1 and $\frac{h}{T} = 48$ (Figure 1, page 6)

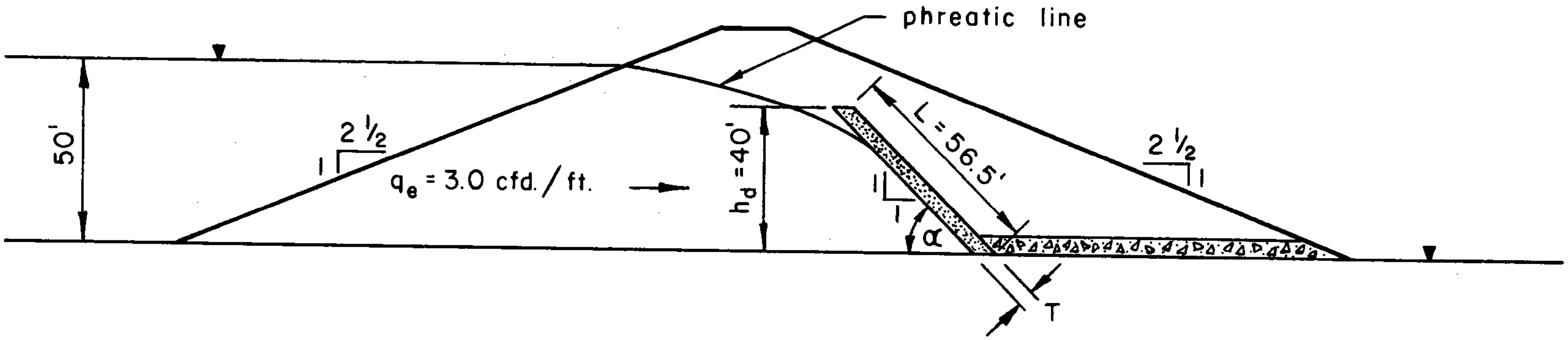
   $T = h/48 = 30/48 = 0.63$ ft.

3. Horizontal thickness = $T/\sin \alpha = 0.63/0.707 = 0.9$ ft.

4. Use horizontal thickness compatible with considerations such as construction methods, anticipated movements, etc., and adjust either the thickness or k_d to provide a reasonable margin of safety, e.g.:

Horizontal thickness	k_d
10 ft.	10 fpd.
5 ft.	20 fpd.

Example C-2: Sloping embankment drain (embankment not susceptible to cracking).



Determine permeability, k_d, required for the sloping embankment drain which has a horizontal thickness of 5 ft. Outlet is adequate. Use Darcy's law.

1. Estimated design discharge is 30 cfd./ft. length of dam. (10 times q_e)

2. $T = 5 \sin \alpha = 5 \times 0.707 = 3.5$ ft.

3. $q_d = k_d iA$ where

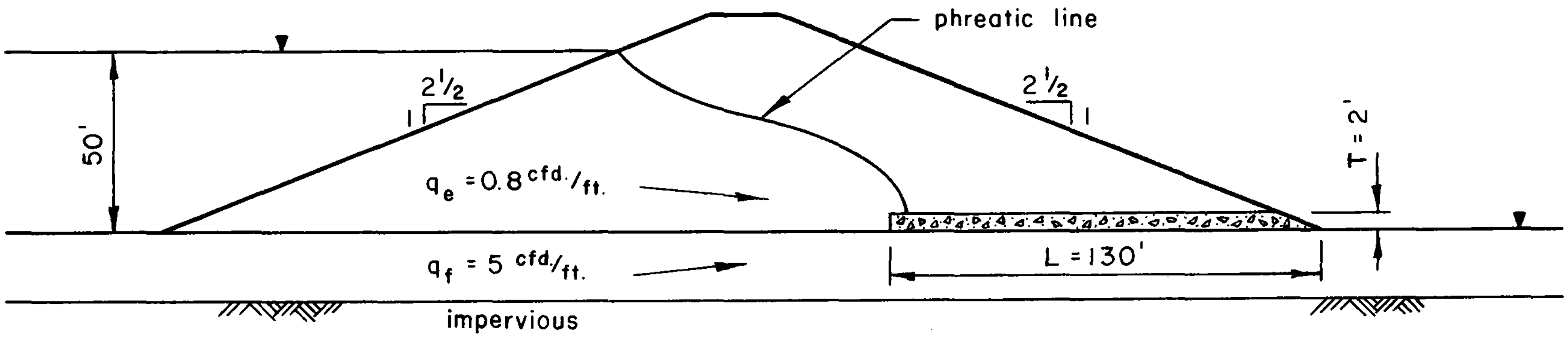
   $i = h/L = 40/56.5 = 0.7$

   $A = T \times 1.0 = 3.5$ sq. ft.

4. $k_d = q_d/iA = 30/(0.7 \times 3.5) = 12$ fpd.

5. Select a drain material with permeability in the range of 10 to 50 fpd.

Example C-3: Blanket Drain



Determine the required permeability of the blanket drain, assuming that outlet at the toe is adequate.

1. Design discharge = (0.8 + 5)10 = 58 cfd. (10 times $q_e + q_f$)

2. $q_d = k_d iA$ where

$$i = \frac{h}{L} = \frac{T}{L} \quad \text{(available head = blanket thickness)}$$

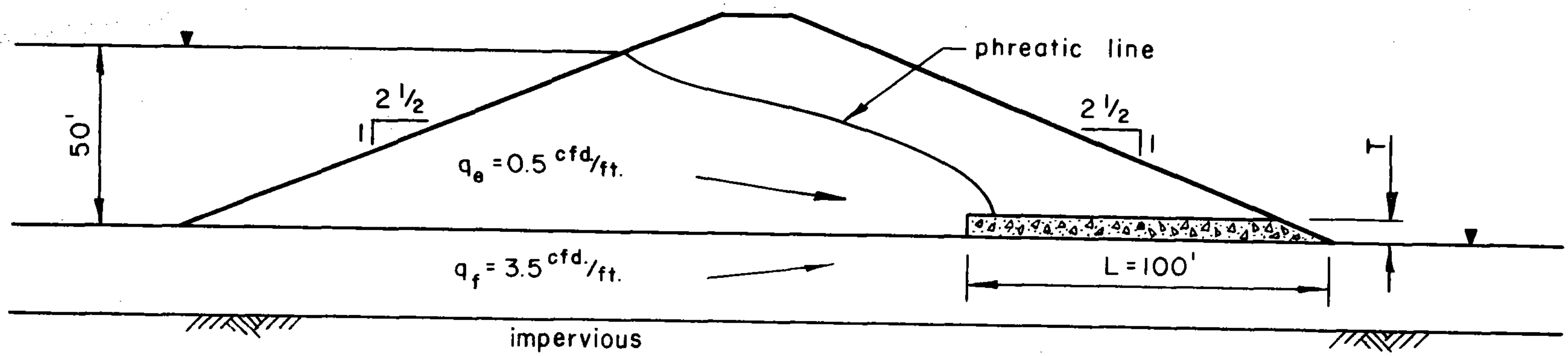
$$A = T \times 1$$

$$q_d = k_d \frac{T^2}{L}$$

3. $k_d = \dfrac{(q_d)(L)}{T^2} = \dfrac{58 \times 130}{4} = 1900$ fpd.

4. Select drain fill with k_d in the range of 2000 fpd.

Example C-4: Blanket Drain



Determine required thickness of blanket drain with k_d = 10 fpd. and outlet at the toe is adequate.

1. Design discharge = 40 cfd./ft. (10 times $q_e + q_f$)

2. $q_d = k_d \frac{T^2}{L}$, $T^2 = \frac{q_d L}{k_d}$

   $T^2 = \frac{40 \times 100}{10} = 400$

   T = 20 ft. (too thick)

3. Try k_d = 100 fpd.

   $T^2 = \frac{40 \times 100}{100} = 40$

   T = 6.3 ft. (too thick)

4. Try k_d = 1000 fpd.

   $T^2 = \frac{40 \times 100}{1000} = 4$

   T = 2 ft.

5. Use T = 2 ft. and k_d in the range of 1000 to 2000 fpd.

Example C-5. Blanket drain.

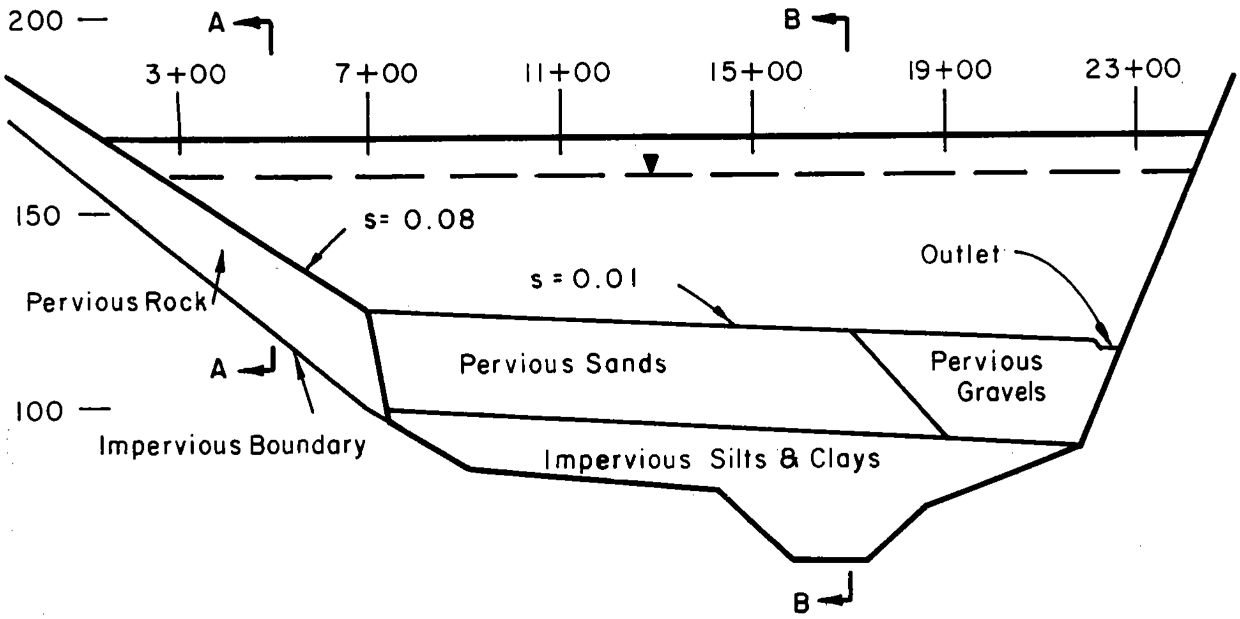


Figure (a). Drain Profile

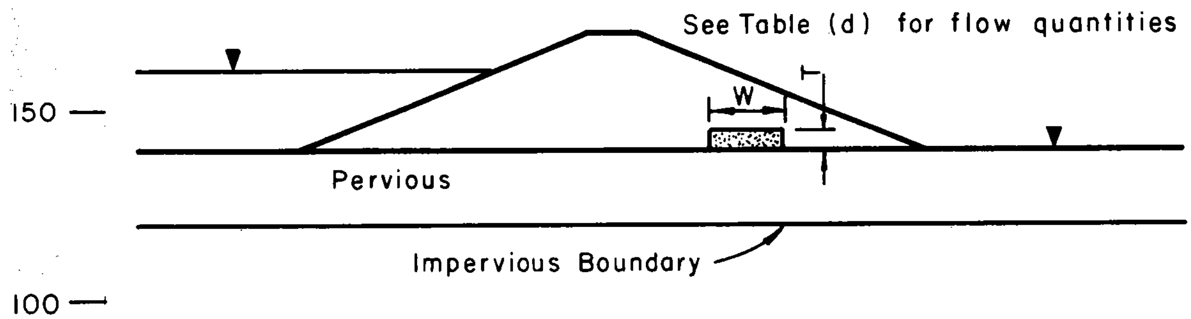


Figure (b). Section A-A

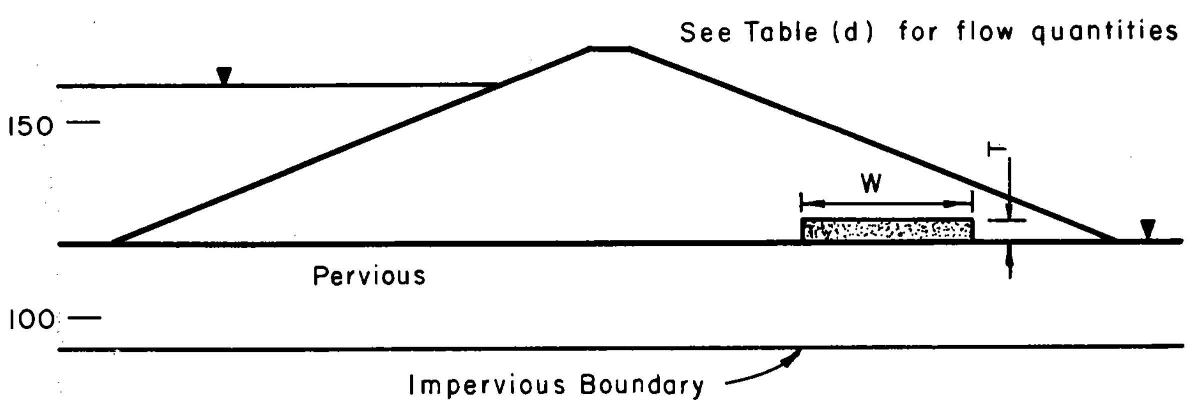


Figure (c). Section B-B

Example C-5 (continued)

Table (d). Flow quantities

Sta.	Dist. ft.	q_e cfd.	q_f per foot	q_{e+f}	q_{e+f} per reach cfd.	q_{e+f} accum. cfd.
3+00		0	2.0	2.0		-
	200				2210	
5+00		0.1	20.0	20.1		2210
	200				6040	
7+00		0.3	40.0	40.3		8250
7+00		0.3	1.0	1.3		8250
	400				520	
11+00		0.3	1.0	1.3		8770
	400				520	
15+00		0.3	1.0	1.3		9290
	400				720	
19+00		0.3	2.0	2.3		10010
	320				736	
22+20		0.3	2.0	2.3		10746

Proportion the blanket drain for the left abutment and flood plain. Permeability of available drain fill is 10,000 fpd. Assume that **gradient**, i, is approximately that of the ground surface, s. All flow is carried across the flood plain to the outlet near Sta. 22+20.

Use $q_d = k_d iA$

$$A = \frac{q_d}{k_d i}$$

Select a reasonable thickness, T, and determine width, W.

$$A = TW$$

Example C-5 (continued)

Table (e). Computations

Sta.	design discharge accum.* (cfd.)	$k_d i$	A sq. ft.	T (assumed) ft.	W ft.	use
3+00	-	800	-	1.0		(15)
5+00	22100	800	28	2.0	14.0	(15)
7+00	82500	800	103	3.0	34.0	(35)
7+00	82500	100	825	4.0	206	
11+00	87700	100	877	4.0	219	
15+00	92900	100	929	4.0	232	
19+00	100100	100	1001	4.0	250	
22+20	107460	100	1075	4.0	269	

Widths from Sta. 7+00 to 22+20 are not reasonable.
Try separate outlet for left abutment. T and W between Sta. 3+00 and 7+00 same as above.

Sta.	design discharge accum.* (cfd.)	$k_d i$	A sq. ft.	T (assumed) ft.	W ft.	use
7+00	-	100	-	2.0		(10)
11+00	5200	100	52	2.0	26	(25)
15+00	10400	100	104	3.0	34.7	(35)
19+00	17600	100	176	4.0	44.0	(45)
22+20	24960	100	250	5.0	50	(50)

See Figure (f) for general layout of this drain.

Other dimensions may be more practical depending on conditions. For instance, width of the abutment portion may need to be large to contact wide spaced bedrock fractures. An additional outlet could be provided to divide flow in the flood plain area.

*(10 times estimated seepage quantities)

Example C-5 (continued)

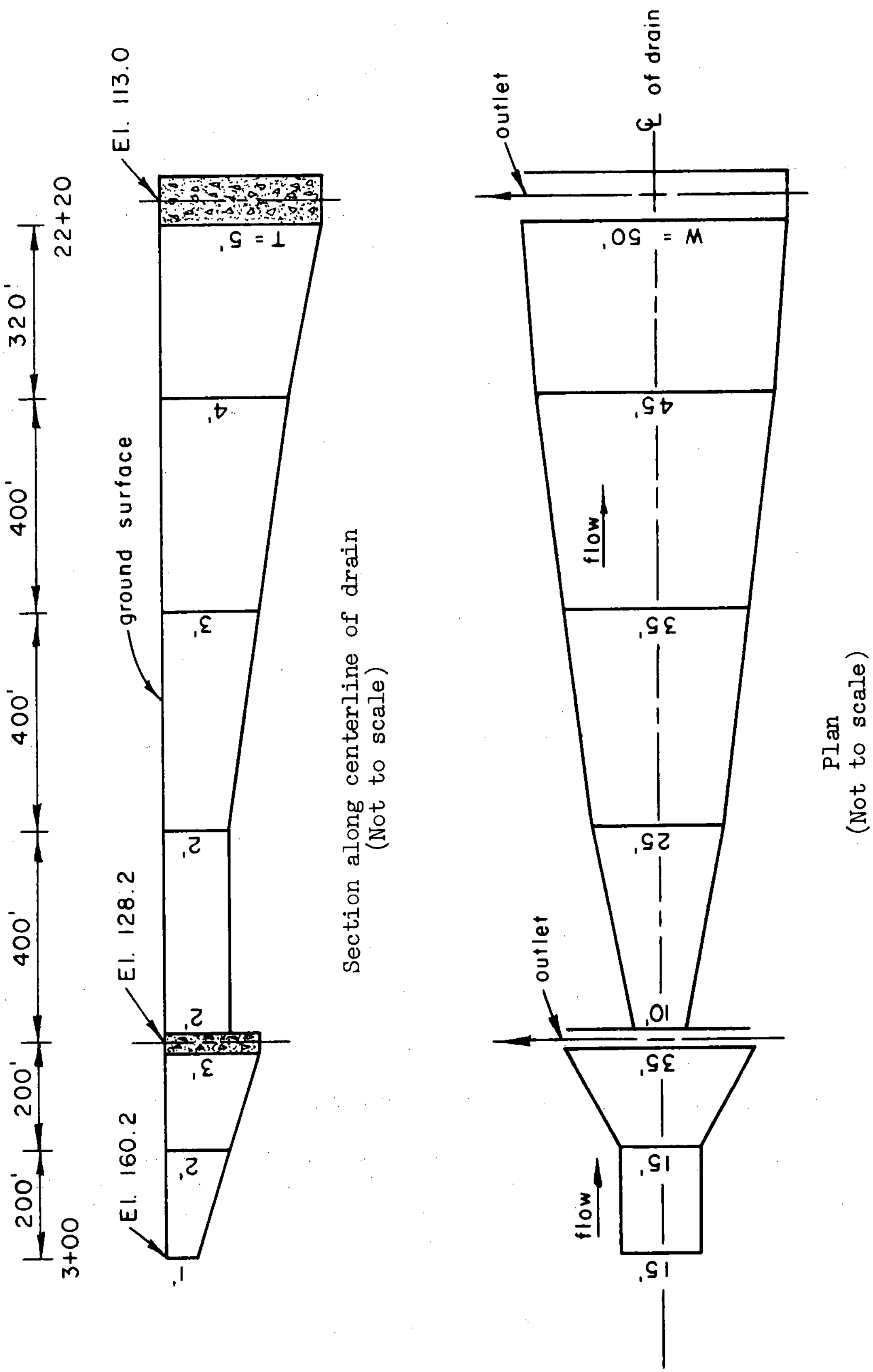


Figure (f). Dimensions of blanket drain.

Example C-6: Foundation trench drain without pipe.

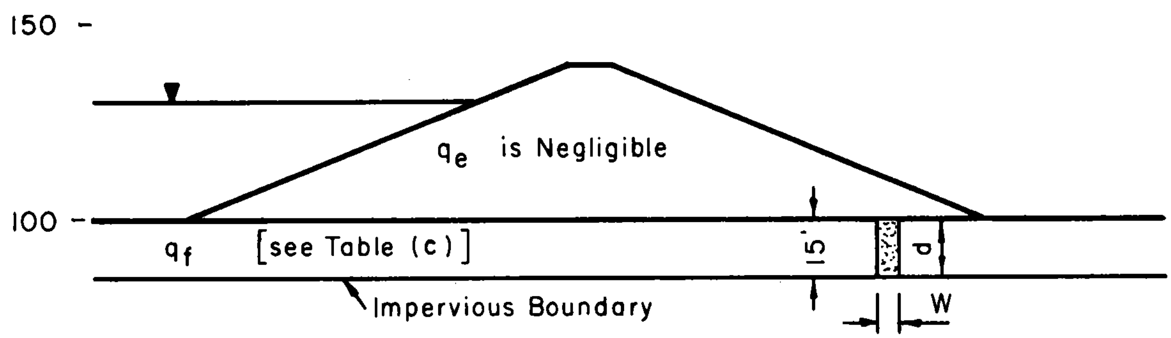


Figure (a). Section A-A.

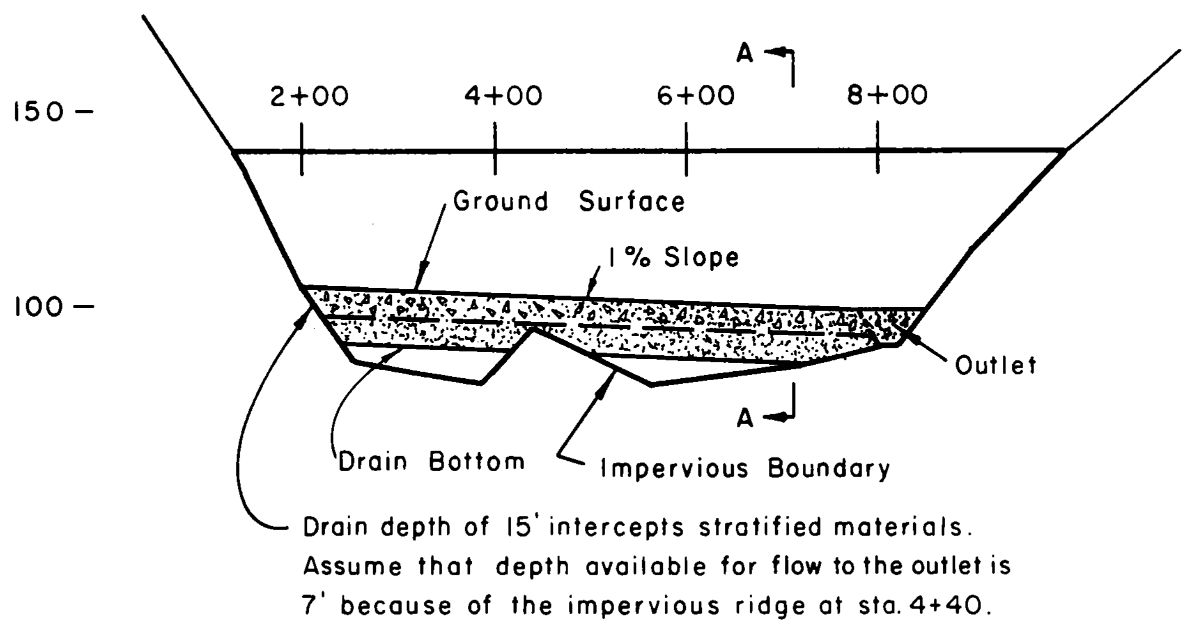


Figure (b). Drain profile.

Table (c). Seepage quantities.

Sta.	Dist. ft.	cfd./ft.	cfd. per reach	cfd. accum.
2 + 00				-
	240	0.8	192	
4 + 40				192
	360	0.4	144	
8 + 00				336

Example C-6 (continued)

Outlet is adequate. Proportion the drain.

1. A filter is needed to prevent migration of base material. Available sand was tested. The coefficient of permeability is 200 fpd. and gradation meets filter requirements.

   Sta. 4+40: $q_d = kiA = 1920$ cfd. (the design discharge)

$$A = \frac{q_d}{ki} = \frac{1920}{200 \times 0.01} = 960 \text{ sq. ft.}$$

$$\text{since } d = 7 \text{ ft., } W = \frac{960}{7} = 137 \text{ ft.}$$

   It is not practical to use the available sand for drain material.

2. Find k required for a drain width of 8 ft. at Sta. 8+00. ($A = 7 \times 8 = 56$ sq. ft.) $q_d = 3360$ cfd. (the design discharge)

$$k = \frac{q_d}{iA} = \frac{3360}{0.01 \times 56} = 6000 \text{ fpd.}$$

3. Find drain width at Sta. 4+40 with k = 6000 fpd.

$$A = \frac{q_d}{ki} = \frac{1920}{6000 \times 0.01} = 32 \text{ sq. ft.}$$

$$W = \frac{32}{7} = 4.6 \text{ (use 5 ft.)}$$

4. Gradation of available gravel is compatible with that of the filter sand and has a coefficient of permeability of 10,000 fpd. Proportion as shown in Fig. (d).

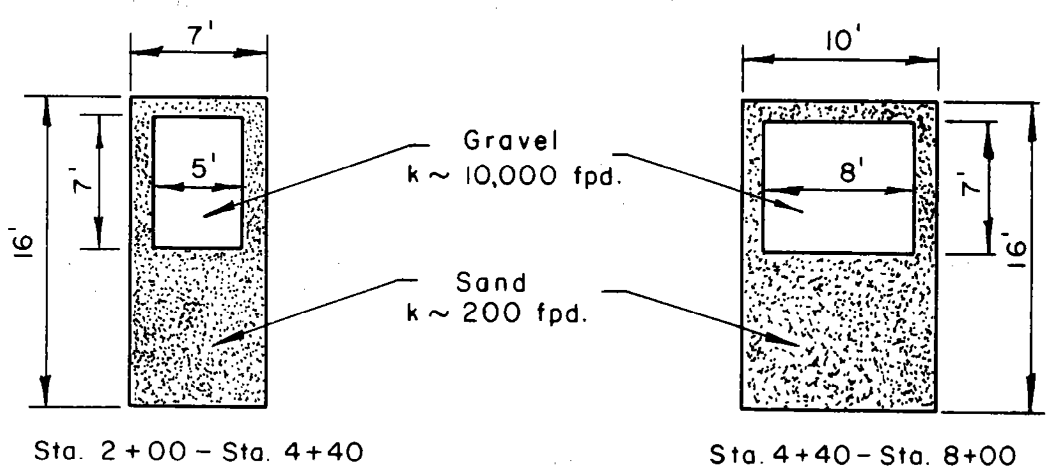


Figure (d). Drain dimensions.

Example C-6 (continued)

Note: Depth increased to 16' to provide space for 1 ft. of filter material over the drain material.

5. Alternates:

   a. Increase flow depth approximately 4 ft. by excavating through the impervious ridge at Sta. 4+40 to reduce width.

   b. Use more than one outlet to reduce width of the drain.

   c. The rectangular drains shown in Figure (d) may be difficult to construct because of the depth. A trapezoidal section could be used to a depth of 8 ft. with a narrow rectangular section to a depth of 16 ft., basing capacity of the system on the area of the trapezoid.

Example C-7: Foundation trench drain with pipe.

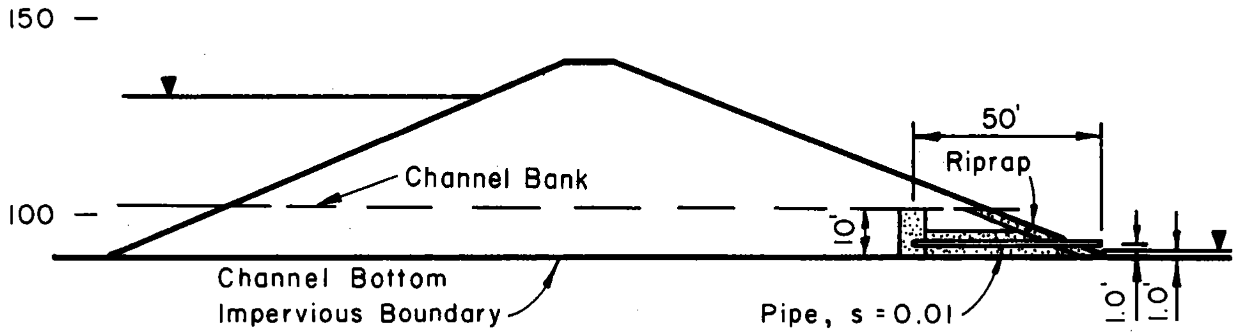


Figure (a). Section A-A.

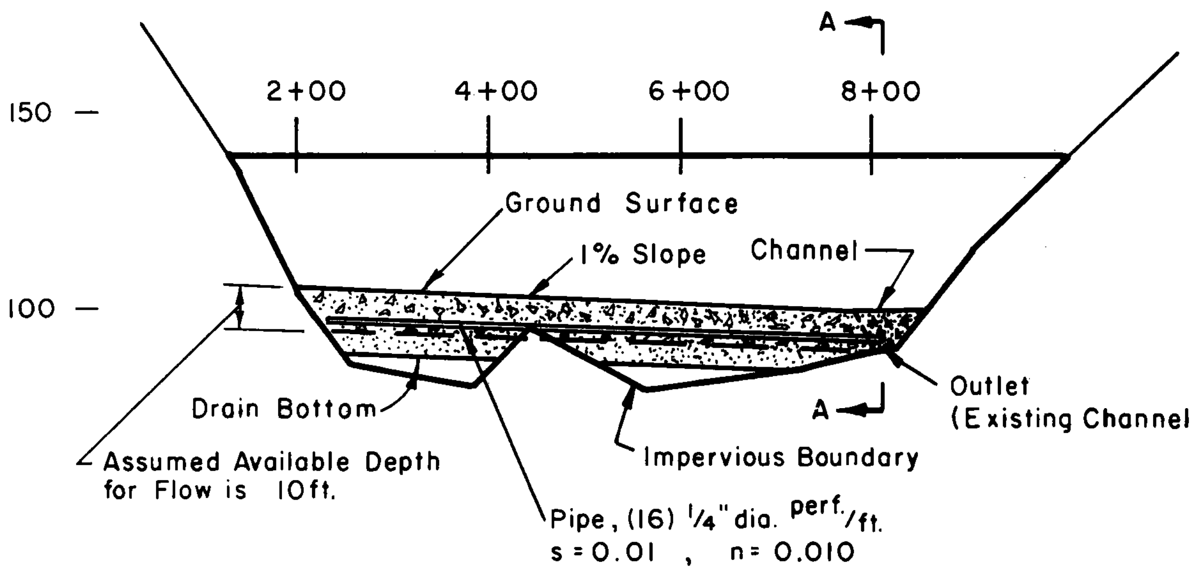


Figure (b). Drain profile.

Proportion the drain for this major structure so that the drain material carries 100% of the design discharge and the pipe carries 100% of the design discharge.

Design discharges are 1920 cfd. at Sta. 4 + 40 and 3360 cfd. at Sta. 8 + 00. Permeability, k, of the drain fill is 10,000 fpd.

Example C-7 (continued)

Determine dimensions of the drain material.

1. Sta. 4+40: q_d = 1920 cfd., k_d = 10,000 fpd., i = 0.01

$$A = \frac{q_d}{k_d i} = \frac{1920}{10{,}000 \times 0.01} = 19.2 \text{ sq. ft.}$$

Since depth = 10 ft., $W = \frac{19.2}{10} = 1.92$ ft. (use 2 ft.)

2. Sta. 8+00: q_d = 3360 cfd., k_d = 10,000 fpd., i = 0.01

$$A = \frac{q_d}{k_d i} = \frac{3360}{10{,}000 \times 0.01} = 33.6 \text{ sq. ft.}$$

Since depth = 10 ft., $W = \frac{33.6}{10} = 3.36$ ft. (use 4 ft.)

Note: The widths (W) in steps 1 and 2 apply to the drain materials only. If filter material is needed, trench widths must be increased. Depth of the drainage system should extend to the drain bottom shown in Figure (b).

3. Outlet. Capacity required is the same as for Sta. 8+00 of the trench drain. Use a section 8 ft. wide and 5 ft. deep which provides the required flow area and should be easy to construct in the old channel. This assumes that the old channel downstream will provide free drainage and not be blocked by subsequent backfilling.

Note: By Darcy's law, capacity of this outlet is adequate with tailwater 10 ft. above outlet channel flow line because slope is 0.01.

$q = kiA = 10{,}000 \times 0.01 \times 40 = 4000$ cfd. compared to inflow of 3360 cfd.

Determine pipe size.

1. Check capacity of perforations assuming that orifice head will not exceed 1.0 ft. Design discharge is 1920 cfd./240 ft. = 8 cfd./ft. (maximum inflow/ft. length of drain).

From Appendix A

Fig. A-3. A = 0.8 sq. in. per ft. with 16, 1/4 in. dia. circular perforations per ft.

Fig. A-4. q = 0.0077 cfs. per ft. = 665 cfd./ft. (> max. inflow) Therefore, specified perforations are adequate.

Example C-7 (continued)

2. Check pipe flow. Max. depth = 3/4 pipe dia. Use ES-97, NEH Section 5.

   Trench drain at Sta. 8+00 and outlet: $s = 0.01$, $s^{1/2} = 0.1$, $n = 0.010$, $q = 3360$ cfd. = 0.039 cfs.

   From ES-97, sheet 3: $\frac{nq}{D^{8/3}s^{1/2}} = 0.422$ for $d/D = 0.75$

   $$D^{8/3} = \frac{0.010 \times 0.039}{0.422 \times 0.1} = 0.00925$$

   $$D = 0.00925^{3/8} = 0.173 \text{ ft. or } 2.08 \text{ in.} \quad (\text{use 4 in. dia.})$$

3. Check flow depth. D = 4 in. or 0.33 ft.

   $$\frac{nq}{D^{8/3}s^{1/2}} = \frac{0.010 \times 0.039}{0.053 \times 0.1} = 0.0735$$

   From ES-97, sheet 3: $d/D = 0.269$

   $d = 0.269 \times 0.33 = 0.089$ ft. or 1.07 in.

   < 3 in. OK

4. A 4-in. dia. pipe is satisfactory for the trench drain and the outlet.

   Note: The design discharges used in these calculations are ten times the estimated seepage quantities (see Example C-6). With both the drain material and the pipe functioning as intended, the system is capable of handling twenty times the estimated seepage quantities. Because of this conservatism, the dimensions of the drain materials might be reduced to 2 ft. by 5 ft. (Sta. 2+00 to 4+40) and 4 ft. by 5 ft. (Sta. 4+40 to outlet). This reduction provides a factor of five for the drain materials and a factor in excess of ten for the pipe.

Example C-8. Fully penetrating blind wells

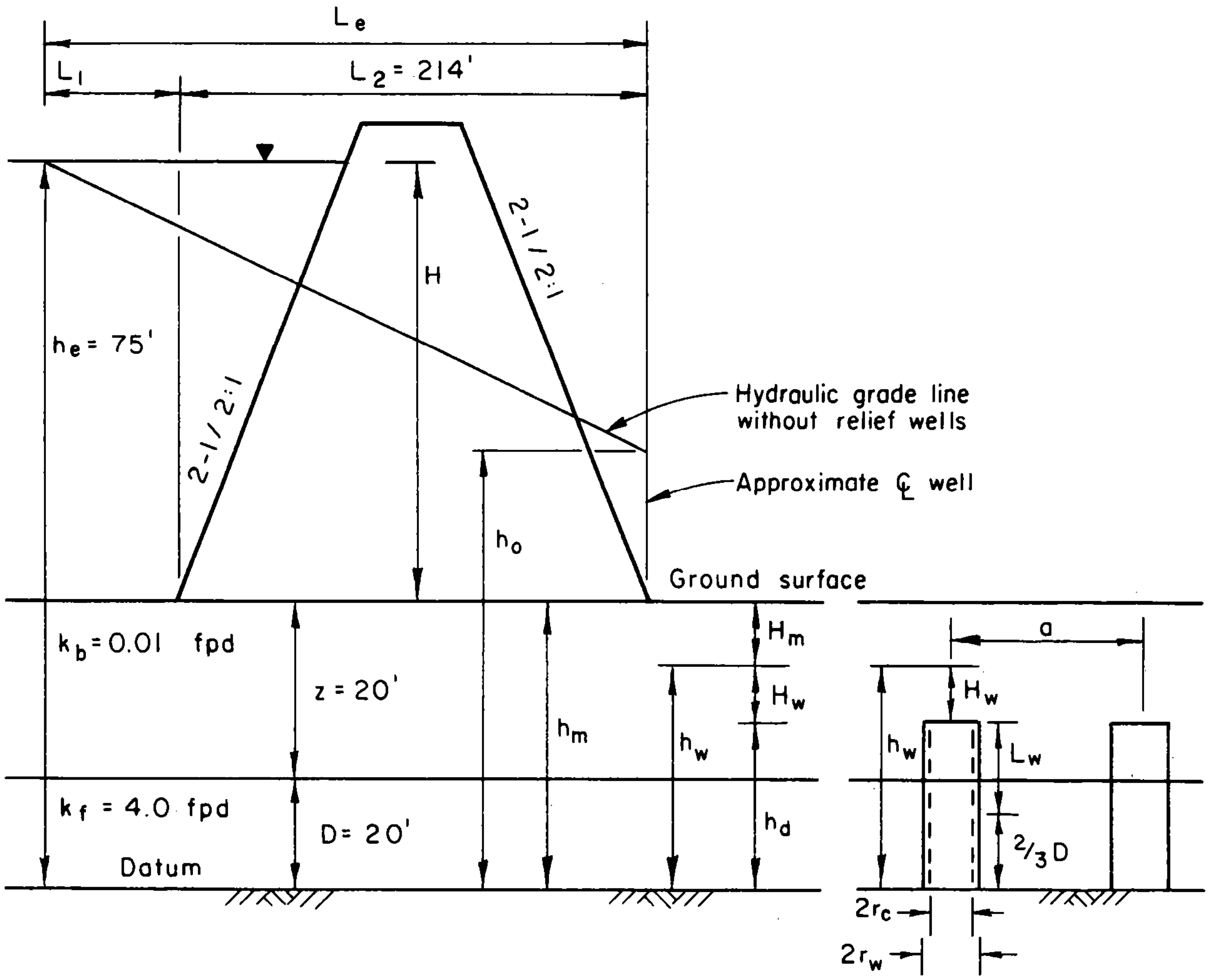


Figure (a). Sketch
(not to scale)

Part I

Determine well spacing if discharge is at elev. 36.0 ft. or higher. Reduce head at toe to ground level or lower. The aquifer is essentially homogeneous and isotropic.

Try 24" dia. wells with 12" dia. drainage core and outer filter (fully penetrating wells).

k_w = 2500 fpd (core)
r_c = 0.5 ft.
A_w = 0.785 sq. ft.
$2r_h$ = 24 in.
H = 35 ft.
h_m = 40 ft.
h_d = 36 ft.

Example C-8 (continued)

Trial No. 1

1. $L_1 = \left(\frac{k_f z D}{k_b}\right)^{1/2} = \left(\frac{4 \times 20 \times 20}{0.01}\right)^{1/2} = 400$ ft.

   $L_e = L_1 + L_2 = 400 + 214 = 614$ ft.

2. $Q_w = k_f \frac{H}{L_e} Da = 4.0 \frac{35}{614} 20a = 4.56a$

3. $L_w = h_d - \frac{2D}{3} = 36 - \frac{2(20)}{3} = 22.67$ ft.

4. $H_w = \frac{Q_w L_w}{k_w A_w} = \frac{(4.56a)(22.67)}{(2500)(0.785)} = 0.0527a$

5. Plot $h_d = 36$ ft. on Figure (b).

6. Plot $h_w = (h_d + H_w)$ vs. a on Figure (b).

a, ft.	H_w, ft.	$h_d + H_w$, ft.
0	0	36
38	2.0	38

7. Plot $h_m = 40$ ft. on Figure (b).

8. $2r_w = \frac{2r_h + 2r_c}{2} = \frac{24 + 12}{2} = 18$ in. (use curve for 20 in.)

9. From Figure B-3 (Appendix B), read H_m for various assumed a values with $H/L_e = 35/614 = 0.057$.

a, ft.	H_m, ft.	$h_m - H_m$, ft.
0	0	40
20	0.4	39.6
30	0.7	39.3
40	1.0	39.0
50	1.3	38.7

   Plot $h_w = (h_m - H_m)$ vs. a on Figure (b).

10. The intersection of the two curves gives a well spacing, a, of 50 ft. and $H_w = 2.6$ ft.

Example C-8 (continued)

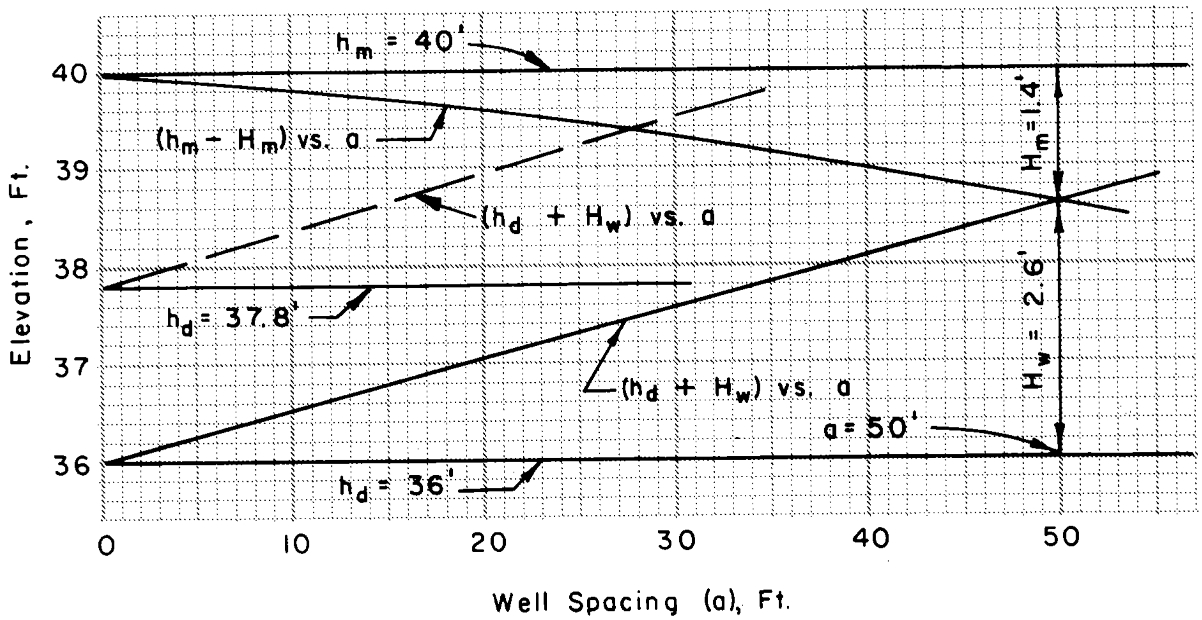


Figure (b). Head loss vs. well spacing

11. From Figure B-8 (Appendix B), the allowable gradient, i_w, is 0.067 for k_w = 2500 fpd and laminar flow. The allowable $H_w = i_w L_w = (0.067)(22.67) = 1.52$ ft. Since this is less than H_w for a = 50 ft., velocity $i_w k_w = Q_w/A_w$ must be reduced. Try increasing h_d which in turn will increase the length of the flow path in the well and decrease the spacing.

Trial No. 2

Approximate h_d by setting $H_w = 0.0527a = 1.52$ ft. (from steps 4 and 11). $a = 1.52/0.0527 = 29$ ft. and from Figure (b), H_m = 0.7 ft. Then $h_d = 40 - 0.7 - 1.5 = 37.8$ ft. and $L_w = 37.8 - 13.3 = 24.5$ ft. The allowable $H_w = (0.067)(24.5) = 1.64$ ft.

12. L_e = 614 ft. (as before)

13. $Q_w = 4.56a$ (as before)

14. $H_w = \dfrac{Q_w L_w}{k_w A_w} = \dfrac{(4.56a)(24.5)}{(2500)(0.785)} = 0.0569a$

15. Plot h_d = 37.8 ft. on Figure (b).

Example C-8 (continued)

16. Plot $h_w = (h_d + H_w)$ vs. a on Figure (b) - dashed line

a, ft.	H_w, ft.	$h_d + H_w$, ft.
0	0	37.8
35	2.0	39.8

17. $h_m = 40$ ft. (as before)

18. $h_m - H_m$ (as before)

19. The intersection of the two curves gives a well spacing of 28 ft. with $H_w = 1.6$ ft.

20. $H_w = 1.6$ ft. < allowable $H_w = 1.64$ ft. (OK)
Use 25 ft. well spacing with discharge at elev. 37.8 ft.

Part II

The required reduction in well velocity also can be achieved by holding the discharge at elevation 36.0 ft. and reducing the head at the toe below ground level by selection of appropriate well spacing.

Try H = 36.9 ft.; then $h_m = 38.1$ ft.
$h_d = 36.0$ ft.

1. $L_e = 614$ ft. (as before)

2. $Q_w = k_f \frac{H}{Le} Da = 4 \frac{36.9}{614} 20a = 4.80a$

3. $L_w = h_d - \frac{2D}{3} = 36.0 - 13.33 = 22.67$ ft.

4. $H_w = \frac{Q_w L_w}{k_w A_w} = \frac{(4.80a)(22.67)}{(2500)(0.785)} = 0.0554a$

5. Plot $h_d = 36.0$ ft. on Figure (c).

6. Plot $h_w = h_d + H_w$ vs. a on Figure (c).

a, ft.	H_w, ft.	$h_d + H_w$, ft.
0	0	36.0
36	2.0	38.0

7. Plot $h_m = 38.1$ ft. on Figure (c).

Example C-8 (continued)

8. From Figure B-3 (Appendix B), read H_m for various assumed a values with $H/L_e = 36.9/614 = 0.060$.

a, ft.	H_m, ft.	$h_m - H_m$, ft.
0	0	38.1
20	0.4	37.7
30	0.7	37.4
40	1.0	37.1
50	1.4	36.7

Plot $h_w = (h_m - H_m)$ vs. a on Figure (c).

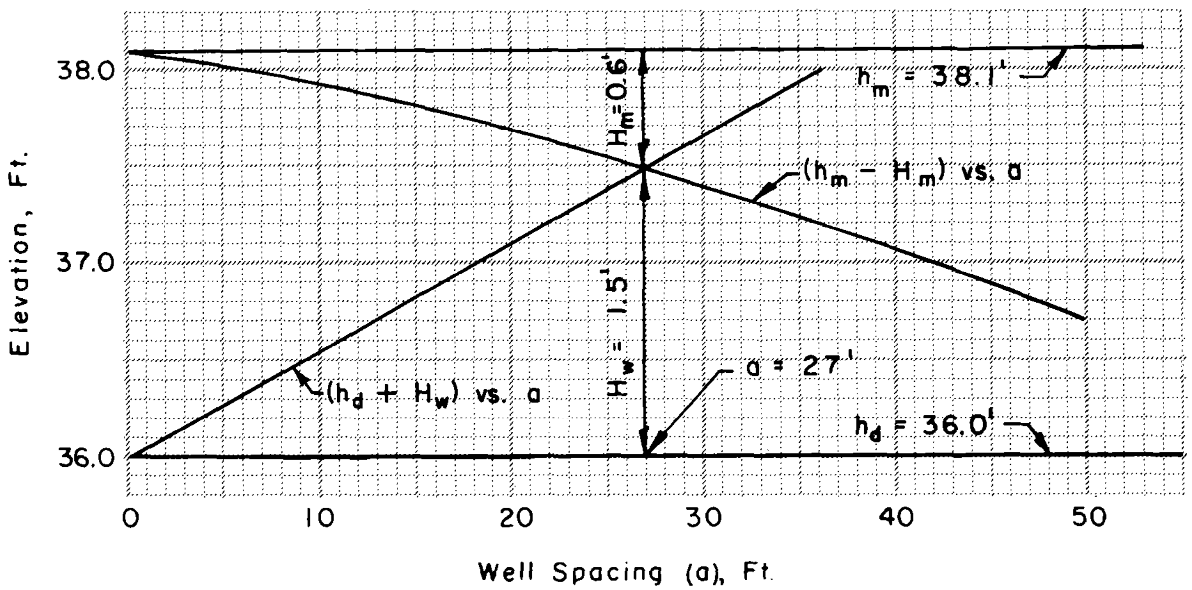


Figure (c). Head loss vs. well spacing

9. Intersection of curves gives a well spacing of 27 ft. and $H_w = 1.50$ ft.

10. $H_w = 1.5$ ft. < allowable $H_w = 1.52$ ft. (OK)
Use a well spacing of 25 ft. with discharge at elev. 36.0.

Part III

Another approach to reduce velocity in the well is to enlarge the drainage core. This should also increase the spacing.

Try 20" diameter core in 30" diameter hole with $h_d = 36.0$ ft., $h_m = 40.0$ ft. and $k_w = 2500$ fpd. $L_w = 22.67$ ft., $r_c = 0.83$ ft., $A_w = 2.182$ sq.ft.

Example C-8 (continued)

1. $L_e = 614$ ft., $H = 35$ ft.

2. $Q_w = 4.56a$ (from Part I)

3. $H_w = \dfrac{Q_w L_w}{k_w A_w} = \dfrac{(4.56a)(22.67)}{(2500)(2.182)} = 0.019a$

4. Plot $h_d = 36.0$ ft. on Figure (d).

5. Plot $h_w = (h_d + H_w)$ vs. a on Figure (d).

a, ft.	H_w, ft.	$h_d + H_w$, ft.
0	0	36
100	1.9	37.9

6. Plot $h_m = 40.0$ ft. on Figure (d).

7. $2r_w = \dfrac{2r_h + 2r_c}{2} = \dfrac{30 + 20}{2} = 25$ in. (use curves for 24 in.)

8. From Figure B-2 (Appendix B), read H_m for various assumed a values with $H/L_e = 35/614 = 0.057$.

a, ft.	H_m, ft.	$h_m - H_m$, ft.
0	0	40.0
20	0.4	39.6
40	1.0	39.0
60	1.7	38.3
80	2.4	37.6
100	3.2	36.8

Plot $h_w = (h_m - H_m)$ vs. a on Figure (d).

9. Intersection of the curves gives a well spacing of 82 ft. with $H_w = 1.55$ ft.

10. $H_w = 1.55$ ft. $\simeq$ allowable $H_w = 1.52$ ft. (OK)
Use well spacing of 80 ft. with discharge at elev. 36.0 ft.

Example C-8 (continued)

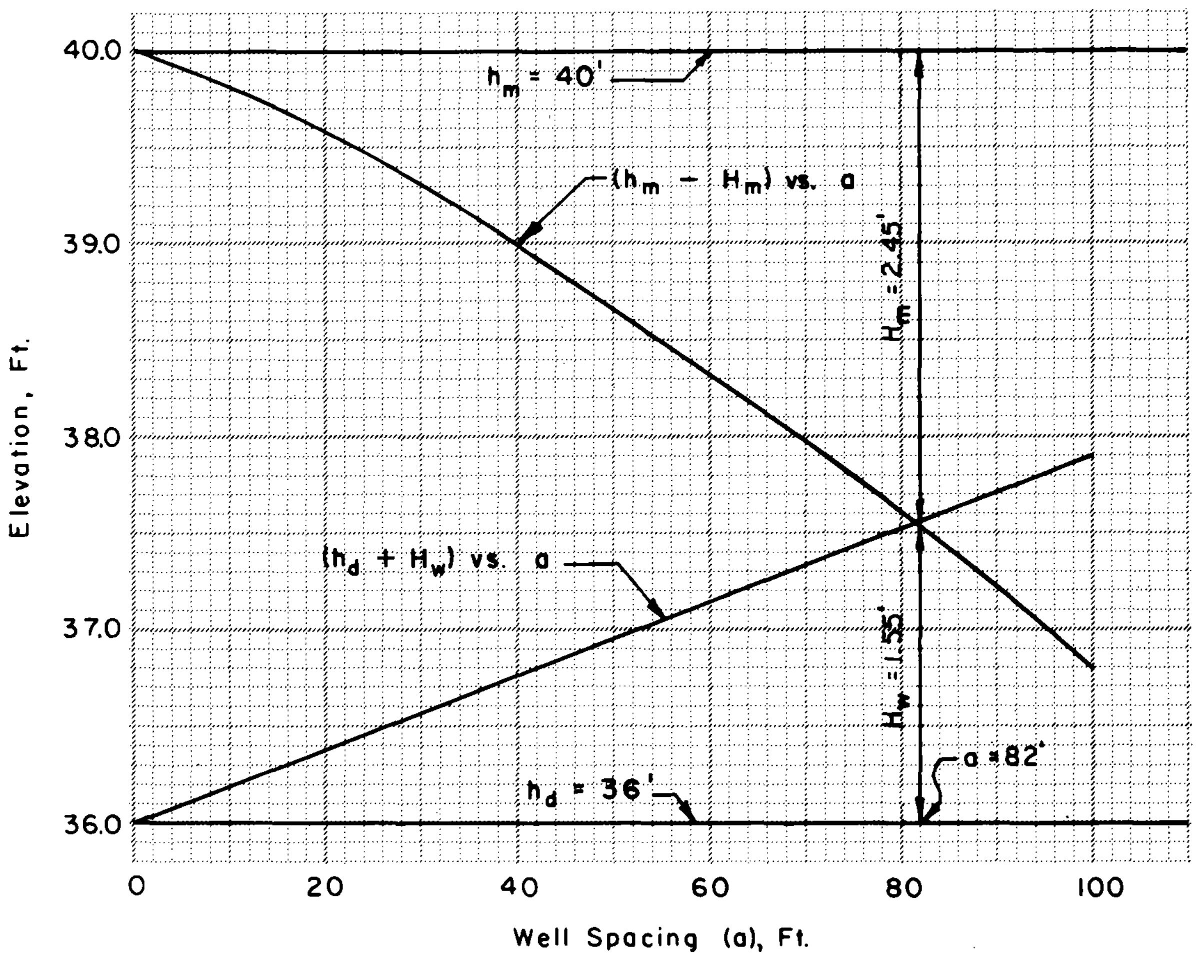


Figure (d). Head loss vs. well spacing

Example C-9. Partially penetrating blind wells

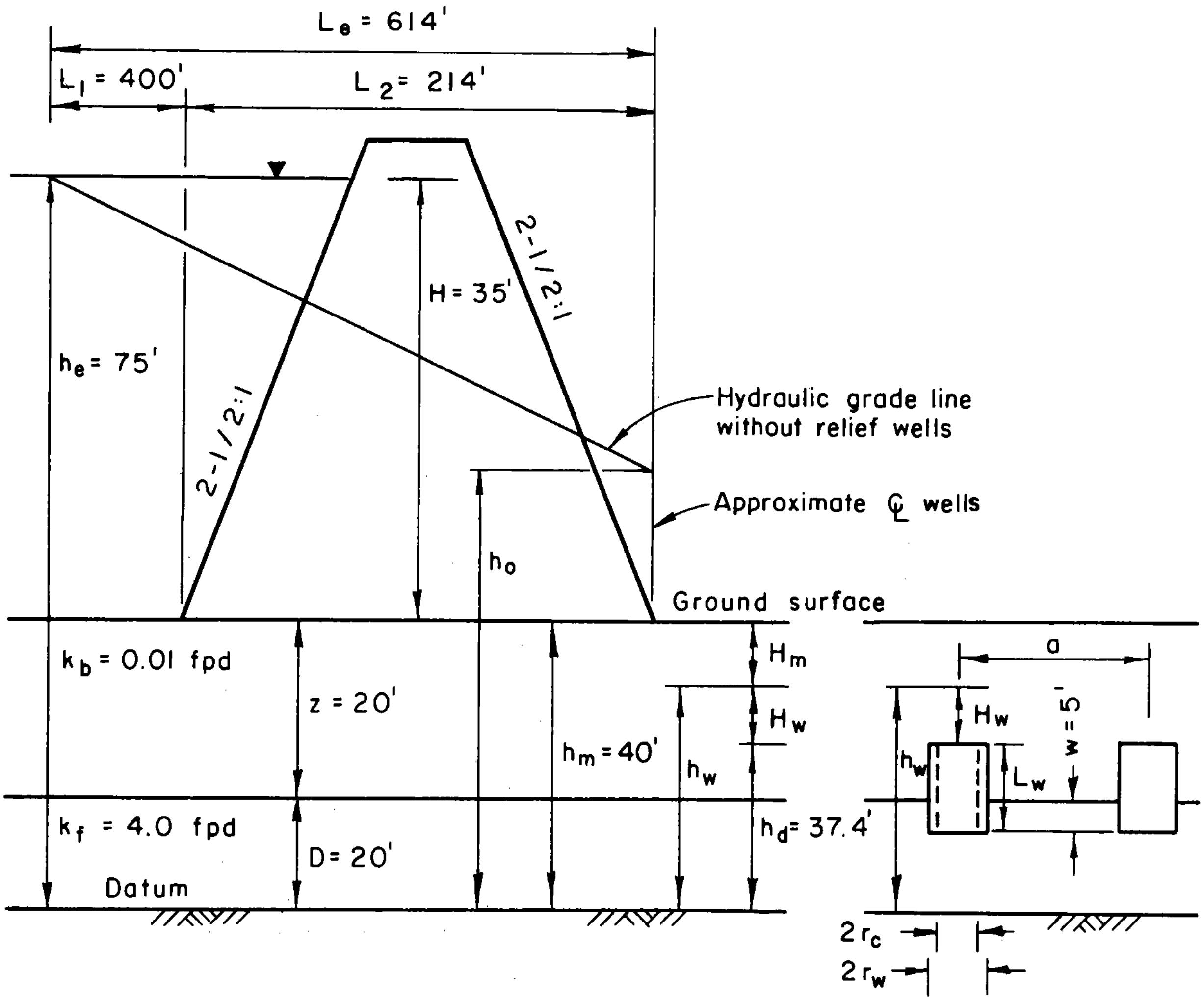


Figure (a). Sketch (not to scale)

Site conditions are the same as used in Example C-8. Consider partially penetrating blind wells with the depth of penetration (w) = 5 ft. and a discharge elevation of 37.4 ft. The aquifer is essentially homogeneous and isotropic.

$2r_h$ = 24", $2r_c$ = 12", k_w = 2500 fpd, A_w = 0.785 sq.ft.

1. H = 35 ft., L_e = 614 ft.

2. $Q_w = k_f \frac{H}{L_e} Da = 4 \frac{35}{614} 20a = 4.56a$

3. $L_w = h_d - (D - w) = 37.4 - (20 - 5) = 22.4$ ft.

Example C-9 (continued)

4. $H_w = \frac{Q_w L_w}{k_w A_w} = \frac{(4.56a)(22.4)}{(2500)(0.785)} = 0.0520a$

5. Plot h_d = 37.4 ft. on Figure (b).

6. Plot ($h_d + H_w$) vs. a on Figure (b).

a, ft.	H_w, ft.	$h_d + H_w$, ft.
0	0	37.4
50	2.6	40.0

7. Plot h_m = 40.0 ft. on Figure (b).

8. $2r_w = \frac{2r_h + 2r_c}{2} = \frac{24 + 12}{2} = 18$ in. (use curve for 20 in.)

9. Plot ($h_m - H_m$) vs. a on Figure (b). (See Example C-8, Part I, step 7.)

10. From Figure B-9 (Appendix B) with w = 5 ft., D = 20 ft., r_w = 0.833 ft., $\frac{r_w}{w}$ = 0.167, and $\frac{w}{D}$ = 0.25, obtain G = 0.717.

11. Read H_m (full penetration) from Figure B-3 (Appendix B) for various a values with H/L_e = 35/614 = 0.057. Correct a to Ga.

a, ft.	H_m, ft.	$h_m - H_m$, ft.	Ga, ft.
0	0	40	0
20	0.4	39.6	14.3
30	0.7	39.3	21.5
40	1.0	39.0	28.7
50	1.3	38.7	35.9

Plot $h_m - H_m$ (full penetration) vs. Ga on Figure (b).

12. Intersection of the ($h_m - H_m$) vs. Ga and ($h_d + H_w$) vs. a curves gives a well spacing of 30 ft. H_w = 1.55 ft.

13. From Example C-8, Part I, allowable H_w = 1.52 ft. (close enough). Use a well spacing of 30 ft. with a discharge elevation of 37.4 ft.

Example C-9 (continued)

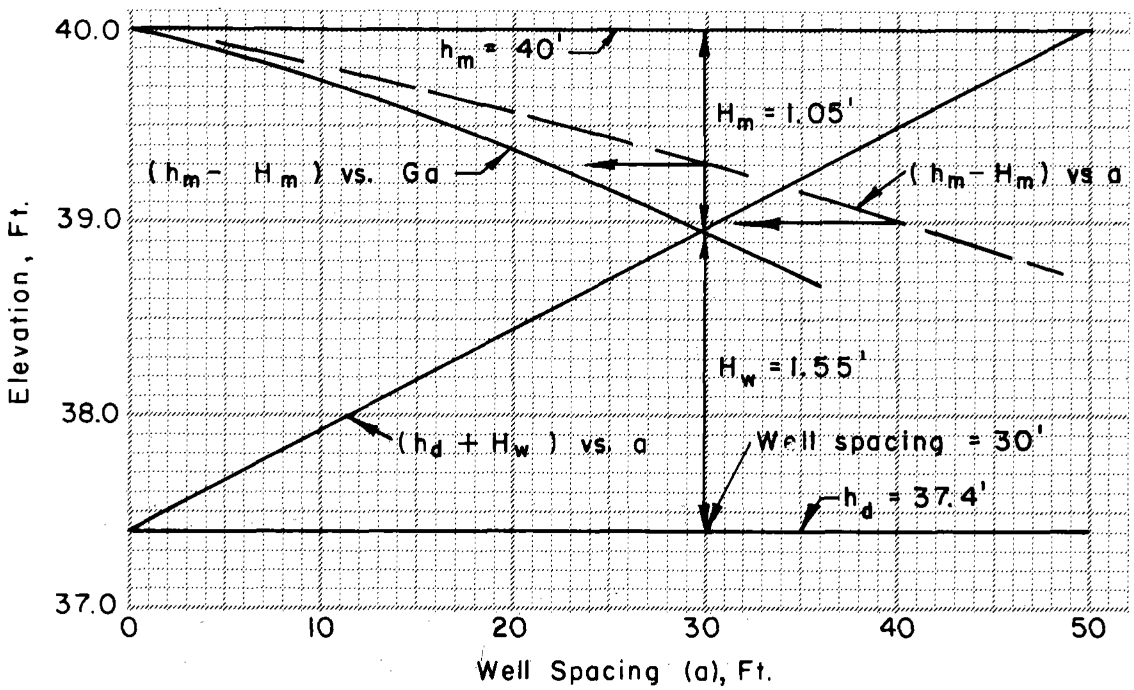


Figure (b). Head loss vs. well spacing

Example C-10. Fully penetrating relief well with screen

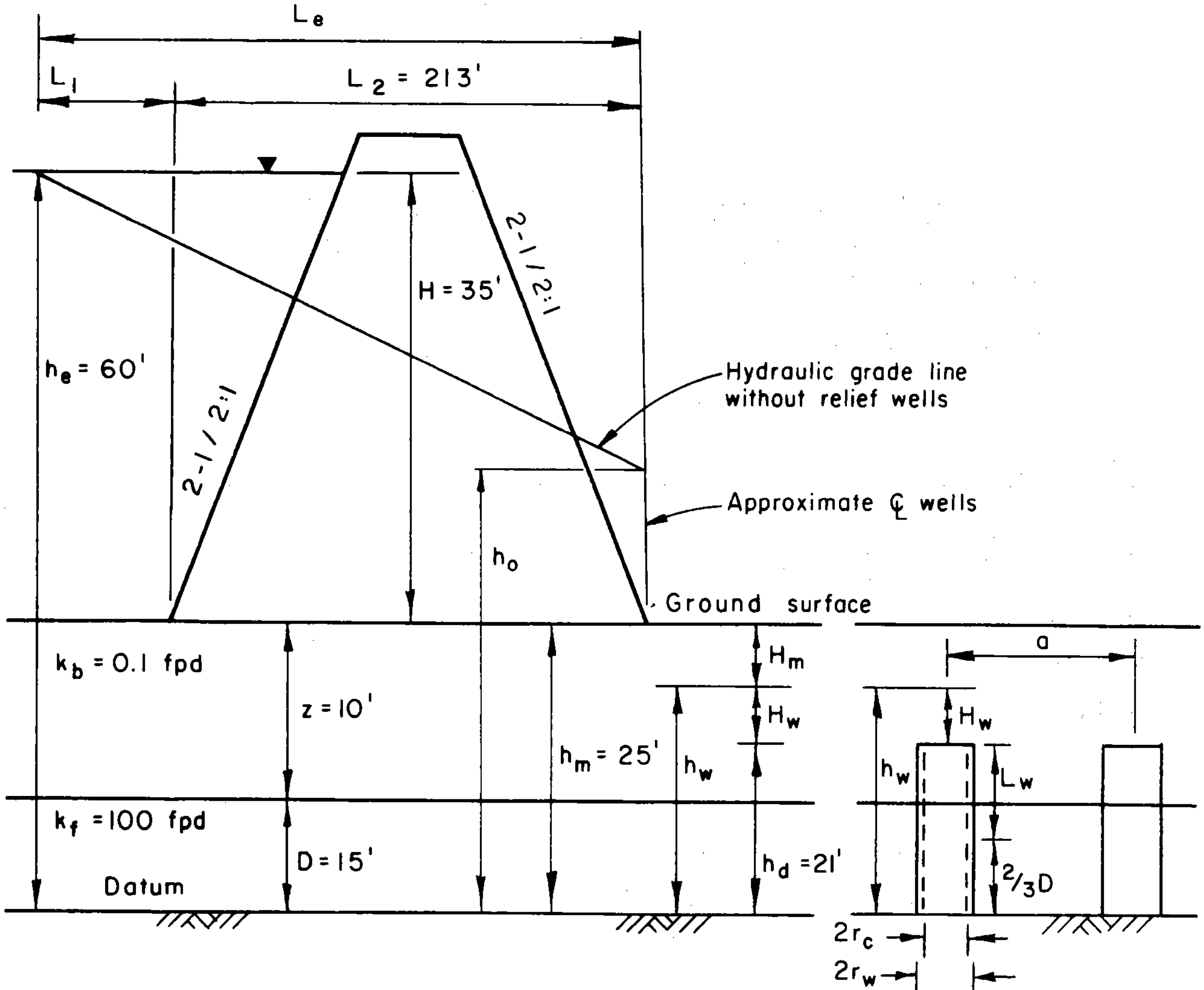


Figure (a). Sketcn (not to scale)

Determine well spacing if discharge is at height 21.0 ft. Reduce head at toe to ground level. Use fully penetrating wells with 4" diameter pipe and 6" filter pack.

$2r_c$ = 4 in., $2r_h$ = 16 in., h_m = 25 ft., h_d = 21 ft.

1. $L_1 = \left[\frac{k_f\ zD}{k_b}\right]^{1/2} = \left[\frac{(100)(10)(15)}{0.1}\right]^{1/2} = 387$ ft.

   $L_e = L_1 + L_2 = 387 + 213 = 600$ ft.

   $H = 35$ ft.

2. $Q_w = k_f \frac{H}{L_e} Da = 100\ \frac{35}{600}\ 15a = 87.5a$. Assume values of a, compute Q_w, and tabulate in table on next page.

Example C-10 (continued)

3. $H_w = H_f + H_v + H_x$ (neglect screen loss by limiting v to 0.1 fps or less)

$L_w = h_d - \frac{2D}{3} = 21 - 10 = 11$ ft.

$H_f = \frac{L_w}{100}(H_{f100}) = \left(\frac{11}{100}\right)(H_{f100}) = 0.11\ H_{f100}$. With C = 100 obtain values of H_{f100} from Figure B-10 (Appendix B) and compute H_f.

Obtain H_v from Figure B-11 (Appendix B).

Considering 4 connections, $H_x = (4)(1.5)(H_v) = 6\ H_v$.

a, ft.	Q_w, cfd	Q_w, gpm	H_{f100}, ft.	H_f, ft.	H_v, ft.	H_x, ft.	H_w	
20	1750	9.1	0.0145	0.0016	--	--	0.002	use
40	3500	18.2	0.051	0.0056	0.0034	0.0204	0.029	(0.03)
60	5250	27.3	0.110	0.012	0.0075	0.045	0.065	(0.07)
80	7000	36.4	0.185	0.020	0.014	0.084	0.118	(0.12)
100	8750	45.0	0.275	0.033	0.02	0.12	0.170	(0.17)

4. Plot h_d = 21.0 ft. on Figure (b).

5. Plot $(h_d + H_w)$ vs. a on Figure (b).

a, ft.	$(h_d + H_w)$, ft.
40	21.03
60	21.07
80	21.12
100	21.17

6. Plot h_m = 25 ft. on Figure (b).

7. $2r_w = \frac{2r_h + 2r_c}{2} = \frac{16 + 4}{2} = 10$ in.

8. Read H_m from Figure B-6 (Appendix B) for various assumed a values with $H/L_e = 35/600 = 0.058$.

a, ft.	H_m, ft.	$(h_m - H_m)$, ft.
20	0.5	24.5
40	1.2	23.8
60	2.1	22.9
80	3.0	22.0
100	4.0	21.0

Plot $(h_m - H_m)$ vs. a on Figure (b).

Example C-10 (continued)

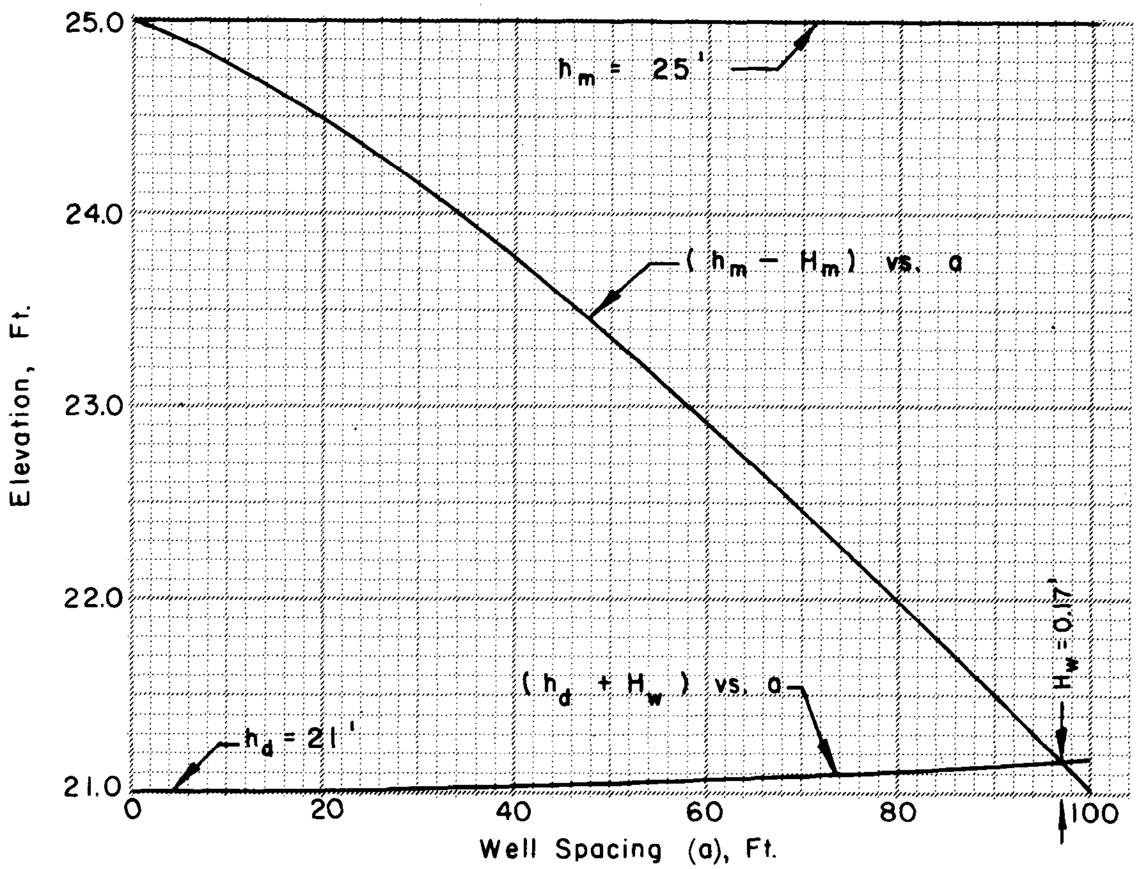


Figure (b). Head loss vs. well spacing

9. Intersection of the curves gives

   a = 97 ft. with H_w = 0.17 ft.

10. Use well spacing of 90 ft.

    Q_w = 87.5 a = (87.5)(90) = 7880 cfd = 0.09 cfs

    Q_w = (A)(v) Limit v to 0.1 fps (Appendix B)

    Unclogged area of screen (A) $= \frac{0.09}{0.1} = 0.9$ ft.^{2} = 130 in.^{2}

    Total screen opening (A_s) $= \frac{A}{0.6} = \frac{130}{0.6} = 217$ in.^{2}

    Well screen length is 14 ft. Select a screen that has at least 217/14 = 15.5 in.^{2} opening per foot length and is compatible with gradation of filter material.

APPENDIX 5C

Water Balance Analysis

Water Balance Analysis

The water balance analysis helps determine if a drainage area is large enough to support a permanent pool during normal conditions. The maximum draw down due to evaporation and infiltration is checked against the anticipated inflows during that same period. The anticipated drawdown during an extended period of no appreciable rainfall is checked as well. This will also help establish a planting zone for vegetation which can tolerate the dry conditions of a periodic draw down of the permanent pool.

The water balance is defined as the change in volume of the permenant pool resulting from the potential total inflow less the potential total outflow.

change volume = inflows outflows

where: *inflows* = *runoff*, *baseflow*, and *rainfall*.
outflows = *infiltration*, *surface overflow*, *evaporation*, and *evapotranspiration*.

This procedure will assume no inflow from baseflow, and because only the permanent pool volume is being evaluated, no losses for surface overflows. In addition, infiltration should be addressed by a geotechnical report. A clay liner should be specified if the analysis of the existing soils indicates excessive infiltration. In many cases, the permeability of clayey soils will be reduced to minimal levels due to the clogging of the soil pores by the fines which eventually settle out of the water column. This may be considered in the water balance equation by assuming the permeability of a clay liner: 1×10^{-6} cm/s (3.94×10^{-7} in/sec.) per specifications. Therefore, the change in storage = runoff evaporation infiltration.

Example

Given:

Drainage Area:	*85 ac.* (Average 65% impervious cover)
SCS RCN:	*72*
Precipitation P (2-year storm):	*3.1* inches
Runoff, Q:	*1.1* inches
Permanent Pool Volume:	*0.65 x 85 ac. = 55 ac.* impervious cover
	WQ volume = *(0.5in.) (55 ac.) (12in./ft.) = 2.29 ac.ft.*
	Retention Basin II (*4* x WQ vol.) = *4 x 2.29 = 9.16 ac.ft.*
Permanent Pool Surface area:	*2.4 ac.*
Infiltration (clay liner per specs.):	1×10^{-6} *cm/s* (3.94×10^{-7} *in/sec.*)

Find:

a) Draw down during highest period of evaporation.
b) Draw down during extended period of no appreciable rainfall.

Solution:

a) Draw down during highest period of evaporation: July

Inflow = Monthly Runoff = *P x E*

Where *P* = precipitation
E = efficiency of runoff (assumed to be ratio of SCS runoff depth to rainfall depth for 2 year storm)
= *1.1 in./ 3.1" = 0.35*

	April	May	June	July	Aug.	Sept.
Precip. (in)	*2.96*	*3.84*	*3.62*	***5.03***	*4.40*	*3.34*
Evap. (in)	*2.28*	*3.89*	*5.31*	***6.23***	*5.64*	*3.92*

(From **Table 5C-1** and **5C-2**)

Inflow: Runoff = *5.03 in. × 0.35 = 1.76 in. = 1.76 in. × 85 ac. 12 in./ft. = 12.5 ac.ft.*

Outflow: Evaporation = *2.4 ac. × 6.23 in. 12 in./ft. = 1.24 ac.ft.*

Infiltration (w/ liner)= *2.4 ac. × (3.94 × 10 ^{7} in./sec.) (3600 sec./hr.) (24 hr./day) (31 days) (12 in./ft.) = 0.21 ac. ft.*

Water balance (w/ liner) = *(inflow) (outflow) = (12.5 ac.ft.) (1.24 + 0.21) ac.ft. = + 11.05 ac.ft.*

Infiltration (w/o liner); assume infiltration rate of .02 in./hr. (clay/silty clay) =
2.4 ac. × .02 in./hr. × (24 hr./day) (31 days) 12in./ft. = 2.97 ac.ft.

Water balance (w/o liner) = *(12.5 ac.ft.) (2.97 + 0.21) ac.ft. = + 9.32 ac.ft.*

b) Drawdown during period of no appreciable rainfall. Assume 45 day period during July and August with no rainfall.

Inflow: runoff *= 0"*

Outflow: Evaporation = Avg. evaporation (July-Aug.) = *6.23 in. + 5.64 in. 2 = 5.93 in.*

Avg. daily evaporation = *5.93 in. 31 days = 0.191 in./day*

Evaporation for 45 days = *45 days × 0.191 in./day = 8.61 in.*

Total evaporation = *2.4 ac. × 8.61 in. 12 in./ft. = 1.7 ac.ft.*

Infiltration (w/ liner): *2.4 ac. × (3.94 x 10^{-7} in./sec.) (3600 sec./hr.) (24 hr./day) (45 days) 12 in./ft. = 0.30 ac.ft.*

Water balance (w/ liner): *(0) (1.7 + 0.30) ac.ft. = 2.0 ac.ft.*

Specify drawdown tolerant plants in areas corresponding to a depth of 2.0 ac.ft. (use stage storage curve).

Infiltration (w/o liner): *2.4 ac.× (.02 in./hr.) (24 hr./day) (45 day) 12 in./ft.*
= 4.32 ac.ft.

Water balance (w/o liner): *(0) (1.7 + 4.32) ac.ft. = 6.02 ac.ft.*

This basin (with out a liner) will experience a significant draw down during drought conditions. Over time, the rate of infiltration may decrease due to the clogging of the soil pores. However, the aquatic and wetland plants may not survive the potential drought conditions and subsequent draw down during the first few years, and eventually give way to invasive species.

Note: A permanent pool volume of *9.16 ac.ft. = 1.29* watershed inches. A rainfall event yielding 1.29" or more of runoff will fill the pool volume.

Table 5C-1
Monthly Precipitation Normals (Inches)

Station	April	May	June	July	August	Sept.
Charlottesville	3.34	4.88	3.74	4.75	4.71	4.10
Danville	3.24	3.85	3.65	4.42	3.80	3.39
Farmville	3.03	4.05	3.41	4.34	3.99	3.18
Fredericksburg	3.05	3.85	3.35	3.65	3.61	3.49
Hot Springs	3.43	4.15	3.36	4.49	3.70	3.39
Lynchburg	3.09	3.91	3.45	4.16	3.59	3.24
Norfolk	3.06	3.81	3.82	5.06	4.81	3.90
Page County	3.84	4.77	4.41	4.50	4.34	4.81
Pennington Gap	4.25	4.83	4.09	4.77	3.76	3.67
Richmond	2.98	3.84	3.62	5.03	4.40	3.34
Roanoke	3.25	3.98	3.19	3.91	4.15	3.50
Staunton	2.82	3.60	2.95	3.49	3.67	3.46
Wash. National Airport	2.31	3.66	3.38	3.80	3.91	3.31
Williamsburg	3.01	4.52	4.03	4.96	4.72	4.25
Winchester	3.08	3.74	3.87	3.89	3.46	3.11
Wytheville	3.09	3.95	3.03	4.20	3.44	3.09

Source: Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

Table 5C-2
Potential Evapotranspiration (Inches) *

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

Source: Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

* Calculated using the Thornthwaite method

APPENDIX 5D

Worksheets

Stage Storage Worksheet

Stage-Storage-Discharge Worksheet

Storage Indication Hydrograph Routing Worksheet; *2S/φt + O Vs. O*

Storage Indication Hydrograph Routing Worksheet

Performance-Based Water Quality Calculations Worksheet 1

Performance-Based Water Quality Calculations Worksheet 2: Situation 2

Performance-Based Water Quality Calculations Worksheet 3: Situation 3

Performance-Based Water Quality Calculations Worksheet 4: Situation 4

Stage-Storage Worksheet

PROJECT: ______________________________________ SHEET _____ OF _____

COUNTY: __________________ COMPUTED BY:_____________ DATE:________

DESCRIPTION: ____

ATTACH COPY OF TOPO: SCALE - 1" = ______ *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME *(ft³)*	*(ac.ft.)*

Stage - Storage - Discharge Worksheet

ELEV *(MSL)*	STORAGE *(ac.ft.)*	WATER QUALITY ORIFICE		2-YEAR CONTROL				10-YEAR CONTROL				RISER STRUCTURE				BARREL				EMERGENCY SPILLWAY		TOTAL *Q* *(cfs)*
		(1)		WEIR (2)		ORIFICE (3)		WEIR (4)		ORIFICE (5)		WEIR (6)		ORIFICE (7)		INLET (8)		OUTLET (9)		(10)		
		h	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*HW/D*	*Q*	*h*	*Q*	*h*	*Q*	

Storage Indication Hydrograph Routing Worksheet
2S/qt + O Vs. O

1	**2**	**3**	**4**	**5**	**6**	**7**
elevation (ft)	*stage (ft)*	*outflow (cfs)*	*storage (cf)*	*2S (cf)*	*2S/qt (cfs)*	*2S/qt + O (cfs)*
from plan	*elev*$_n$ *- elev*$_o$	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /qt of hydrograph*	*Col 3 + Col 6*

Storage Indication Hydrograph Routing Worksheet

1	2	3	4	5	6	7
n	*Time* *(min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n/\Delta t - O_n$ *(cfs)*	$2S_{n+1}/\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n + *Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n

Worksheet 1
Page 1 of 3

STEP 1 **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

Applicable area (A)* = ________ acres

Post-development impervious cover:

structures =__________acres

parking lot =__________acres

roadway =__________acres

other:

______________=__________acres

______________=__________acres

Total = __________acres

I_{post} = (total post-development impervious cover ÷ A) × 100 = ________%

* The area subject to the criteria may vary from locality to locality. Therefore, consult the locality for proper determination of this value.

STEP 2 **Determine the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$).**

Average land cover condition ($I_{watershed}$):
If the locality has determined land cover conditions for individual watersheds within its jurisdiction, use the watershed specific value determined by the locality as $I_{watershed}$.

$I_{watershed}$ = ________%

Otherwise, use the Chesapeake Bay default value:

$I_{watershed}$ = 16%

Worksheet 1

Page 2 of 3

Existing impervious cover ($I_{existing}$):

Determine the existing impervious cover of the development site if present.

Existing impervious cover:

structures = __________acres

parking lot = __________acres

roadway = __________acres

other:

______________=__________acres

______________=__________acres

Total = __________acres

$I_{existing}$ = (total existing impervious cover ÷ A*) × 100 = _______%

* The area should be the same as used in STEP 1.

STEP 3 **Determine the appropriate development situation.**

The site information determined in STEP 1 and STEP 2 provide enough information to determine the appropriate development situation under which the performance criteria will apply. Check ( ) the appropriate development situation as follows:

_____ **Situation 1**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than or equal to the average land cover condition ($I_{watershed}$).

$\mathbf{I_{post}}$ ________% $\mathbf{I_{watershed}}$ ________%

Worksheet 1

Page 3 of 3

_____ **Situation 2**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

$\mathbf{I_{existing}}$ ________% $\mathbf{I_{watershed}}$ ________%; **and**

$\mathbf{I_{post}}$ ________% > $\mathbf{I_{watershed}}$ ________%

_____ **Situation 3**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

_____ **Situation 4**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. If the proposed development meets the criteria for development Situations 2, 3, or 4, then proceed to STEP 4 on the appropriate worksheet.

Worksheet 2 : Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ =________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% $\mathbf{I_{watershed}}$ ________%; **and**

$\mathbf{I_{post}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load ($\mathbf{L_{pre}}$).**

$\mathbf{L_{pre(watershed)}}$ = [0.05 + (0.009 × $I_{watershed}$)] × A × 2.28 (**Equation 5-16**)

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(watershed)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 2 : Situation 2

Page 2 of 4

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28 \quad (\textbf{Equation 5-21})$$

where:
- L_{post} = relative post-development total phosphorous load (pounds per year)
- I_{post} = post-development percent impervious cover (percent expressed in whole numbers)
- A = applicable area (acres)

$\mathbf{L_{post}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

STEP 6 **Determine the relative pollutant removal requirement (RR).**

RR = L_{post} $L_{pre(watershed)}$

RR = ________ ________

= __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

$$\mathbf{EFF} = (RR \div L_{post}) \times 100 \quad (\textbf{Equation 5-22})$$

where:
- EFF = required pollutant removal efficiency (percent expressed in whole numbers)
- RR = pollutant removal requirement (pounds per year)
- L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

Worksheet 2 : Situation 2

Page 3 of 4

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where:

L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)

A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

Worksheet 2 : Situation 2

Page 4 of 4

4. Calculate the pollutant load removed by the proposed BMP(s):

$$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP} \qquad \textbf{(Equation 5-24)}$$

where: $L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = ___________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = ___________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = ___________ pounds per year

5. Calculate the total pollutant load removed by the BMP(s):

$$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots \qquad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = ___________ + ___________ + ___________ + . . .

= ___________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}}$ **RR**

___________ ___________

Worksheet 3 : Situation 3

Page 1 of 5

Summary of Situation 3 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ =________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

1. Pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$ **(Equation 5-17)**

where: $L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= ___________ pounds per year

<u>Worksheet 3 : Situation 3</u>

Page 2 of 5

2. Pre-development pollutant load based on the average land cover condition:

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$ (**Equation 5-16**)

where:

$L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)

$I_{watershed}$ = average land cover condition for specific watershed or locality **<u>or</u>** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

<u>STEP 5</u> **Determine the relative post-development pollutant load ($\mathbf{L_{post}}$).**

$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$ (**Equation 5-21**)

where:

L_{post} = relative post-development total phosphorous load (pounds per year)

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L_{post}} = [0.05 + (0.009 \times$ ______$)] \times$ ______ $\times 2.28$

= __________ pounds per year

<u>STEP 6</u> **Determine the relative pollutant removal requirement (RR).**

$\mathbf{RR} = L_{post} \quad (0.9 \times L_{pre(existing)})$

= ________ (0.9 × ________) = __________ pounds per year

<u>or</u>

$\mathbf{RR} = L_{post} \quad L_{pre(watershed)}$

= ________ ________ = __________ pounds per year

Worksheet 3 : Situation 3

The pollutant removal requirement (RR) for Situation 3 is the lesser of the two values calculated above:

RR = __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

$$\textbf{EFF} = (RR \div L_{post}) \times 100 \qquad \textbf{(Equation 5-22)}$$

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

Worksheet 3 : Situation 3

Page 4 of 5

3. Determine the pollutant load entering the proposed BMP(s):

$$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28 \qquad \textbf{(Equation 5-23)}$$

where: L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= ___________ pounds per year

$\mathbf{L_{BMP2}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= ___________ pounds per year

$\mathbf{L_{BMP3}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= ___________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP} \qquad \textbf{(Equation 5-24)}$$

where: $L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = ___________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = ___________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = ___________ pounds per year

Worksheet 3 : Situation 3

Page 5 of 5

5. Calculate the total pollutant load removed by the BMP(s):

$$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots \quad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}}$ **RR**

__________ __________

Worksheet 4 : Situation 4

Page 1 of 6

Summary of Situation 3 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A) = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ =________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

Summary of existing BMP:

Existing BMP drainage area ($\mathbf{A_{existBMP}}$) = ________ acres

$\mathbf{I_{pre(BMP)}}$ = (total pre-development impervious cover ÷ $A_{existBMP}$) × 100 = ________%

$\mathbf{EFF_{existBMP}}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

STEP 4 **Determine the relative pre-development pollutant load ($\mathbf{L_{pre}}$).**

1. Calculate pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × $I_{existing}$)] × A × 2.28 **(Equation 5-17)**

where:
$L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 4 : Situation 4
Page 2 of 6

2. Calculate pre-development pollutant load to existing BMP:

$$\mathbf{L_{pre(BMP)}} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28 \qquad \textbf{(Equation 5-18)}$$

where: $L_{pre(BMP)}$ = relative pre-development total phosphorous load to existing BMP (pounds per year)
$I_{pre(BMP)}$ = existing impervious cover to existing BMP (percent expressed in whole numbers)
$A_{existBMP}$ = drainage area of existing BMP (acres)

$L_{pre(BMP)}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= ___________ pounds per year

3. Calculate pre-development pollutant load removed by existing BMP:

$$\mathbf{L_{removed(existingBMP)}} = L_{pre(BMP)} \times EFF_{existBMP} \qquad \textbf{(Equation 5-19)}$$

where: $L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP (pounds per year)
$L_{pre(BMP)}$ = relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)
$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$\mathbf{L_{removed(existingBMP)}}$ = _________ × _________

= ___________ pounds per year

Steps 2 and 3 are repeated for each existing BMP on the site.

Worksheet 4: Situation 4

Page 3 of 6

4. Calculate the pre-development pollutant load while being served by existing BMP(S):

$$\mathbf{L}_{\mathbf{pre(existingBMP)}} = L_{pre(existing)} \quad (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20

where:

$L_{pre(existingBMP)}$ = relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)

$L_{pre(existing)}$ = relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)

$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)

$\mathbf{L}_{\mathbf{pre(existingBMP)}}$ = _________ (_________ + _________ + _________)

= ____________ pounds per year

STEP 5 **Determine the relative post-development pollutant load ($\mathbf{L}_{\mathbf{post}}$).**

$\mathbf{L}_{\mathbf{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$ (**Equation 5-21**)

where:

L_{post} = relative post-development total phosphorous load (pounds per year)

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L}_{\mathbf{post}}$ = [0.05 + (0.009 × ______)] × ______ × 2.28

= __________ pounds per year

STEP 6 **Determine the relative pollutant removal requirement (RR).**

RR = $L_{post} \quad L_{pre(existingBMP)}$

= ________ ________

RR = __________ pounds per year

Worksheet 4 : Situation 4

Page 4 of 6

2. Pre-development pollutant load based on the average land cover condition:

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$ (**Equation 5-16**)

where:
$L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

$\mathbf{EFF} = (RR \div L_{post}) \times 100$ **(Equation 5-22)**

where:
EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3: ____

Worksheet 4 : Situation 4
Page 5 of 6

3. Determine the pollutant load entering the proposed BMP(s):

$$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28 \qquad \textbf{(Equation 5-23)}$$

where:
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP2}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP3}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP} \qquad \textbf{(Equation 5-24)}$$

where:
$L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

Worksheet 4 : Situation 4

Page 6 of 6

5. Calculate the total pollutant load removed by the existing and proposed BMP(s):

$$\mathbf{L}_{\mathbf{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + $$

$$+ \; L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)} \quad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1, **Equation 5-24**
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2, **Equation 5-24**
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3, **Equation 5-24**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 1, **Equation 5-19**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 2, **Equation 5-19**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 3, **Equation 5-19**

$\mathbf{L}_{\mathbf{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L}_{\mathbf{removed/total}}$ **RR**

__________ __________